Strengthening of Concrete Members with Advanced Composite Materials

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Abstract

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Rehabilitation and strengthening have become a major focal point for the Nation's aging infrastructure. An increasing number of reinforced concrete structures, particularly bridges and parking garages, are in deteriorated state and need to be repaired and strengthened. Traditionally, strengthening of existing structures relied on addition of external reinforcement in the form of reinforcing steel cables or plates. However, the use of these components requires continuous maintenance and protection against corrosion along the life time of the structures. The use of fiber reinforced plastic (FRP) materials in place of conventional steel in concrete structures has been growing rapidly in recent years. The non-corrosive characteristics of these materials offer a promising solution to durability problems caused by corrosion of steel. In addition, the high strength-to-weight ratio and the good fatigue properties of these materials can improve the service life and the load carrying capacity of concrete structures.

This research involves two experimental investigations into the behaviour of prestressed concrete members strengthened using two typical techniques: the first is by application of external post-tensioning using carbon fiber reinforced plastic (CFRP) cables, whereas the second is by bonding composite straps on the tension side of the members. In the first investigation, a total of twelve partially prestressed concrete beams were tested. Nine of the beams were first subjected to loads large enough to cause considerable cracking and deformations and then strengthened with the external CFRP cables and loaded up to

failure. The CFRP cables used were CFCC 1×7 - 5 mm and 7.5 mm diameter, manufactured by Tokyo Rope, Japan. The remaining three beams were loaded monotonically from zero up to failure without strengthening and were used as control beams for comparison purposes. The twelve beams had the same concrete cross-section dimensions and were divided into three groups; each group consisted of four beams of the same length but with different levels of internal prestressing. The purpose was to study the effects of the span-to-depth ratio and the partial prestressing ratio or the reinforcing index on the performance of reinforced or prestressed concrete beams after being strengthened with the external CFCC cables. The experiments showed an enhancement of the overall performance of the beams and an increase up to 70% in their ultimate resistance after strengthening.

In the second investigation, Polyester straps supplied by Caristrap International Inc. were used to retrofit and improve the strength and serviceability of railway crossties. Two full scale prestressed concrete railway crossties with different levels of prestressing were loaded upward to produce cracking on the top surface similar to that caused by rebound forces due to wheel impact. The ties were then retrofitted by bonding two layers of the straps to the top surface using epoxy adhesive and then reloaded up to failure. The experimental results were compared with those obtained from an extensive study previously conducted at Concordia University on the performance of the crossties without strengthening. The strength of the retrofitted ties was found to be 40% to 48% higher than those tested without straps.

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Dedications

To my father who guided me in establishing lifelong goals and set the pace for me

To my mother, my hero who gives and loves more unconditionally than any other person I know.

To my wife, my best friend and companion.

To my sisters, who always believed in me.

To my nephew and niece for always bringing joy to our lives.

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Dimensions

h = total height of the section

b = width of section

d_p = depth from concrete extreme compressive fiber to centroid of prestressing steel

d_{pe} = depth from concrete extreme compressive fiber to centroid of external prestressing cables

d_e = depth from concrete extreme compressive fiber to centroid of tensile forces

d_s = depth from concrete extreme compressive fiber to centroid of nonprestressed tensile steel

d_s' = depth from concrete extreme compressive fiber to centroid of nonprestressed compressive steel

c = neutral axis depth

a = depth of the equivalent rectangular stress block

e_m = eccentricity of the tendon at midspan

e_s = eccentricity of the tendon at anchorage ends

L = span between the supports

S = span between the end anchorages

A_c = cross-sectional area of concrete

A_{ps} = area of prestressing steel

A_s = area of nonprestressed tensile steel

A_s' = area of nonprestressed compressive steel

I_g = moment of inertia of gross cross-section

Z = section modulus

Stresses

E_c = modulus of elasticity of concrete

E_s	=	modulus of elasticity of nonprestressed tensile steel
E_s'	=	modulus of elasticity of nonprestressed compressive steel
E_{ps}	=	modulus of elasticity of prestressing steel
E_{FRP}	=	modulus of elasticity of the FRP tendon
f_c'	=	concrete compressive strength
f_{ps}	=	stress in the prestressing steel at the ultimate flexural strength
f_{peff}	=	effective stress in prestressing steel
f_{pu}	=	ultimate strength of the prestressing steel
f_{py}	=	yield strength of the prestressing steel
f_y	=	yield strength of nonprestressed tensile steel
f_y'	=	yield strength of nonprestressed compressive steel
Δf_p	=	increase in the stress in the prestressing tendon
$\Delta f_{p(ext)}$	=	increase in stress in the external prestressed cable

Moments and Forces

M_{cr}	=	cracking Moment
M_u	=	ultimate flexural strength
P	=	total applied load
P_u	=	ultimate applied load
P_{cr}	=	cracking load
ΔP	=	increment of applied load
C_c	=	compressive force in concrete section
C_s	=	compressive force in nonprestressed compressive steel
T_s	=	tension force in nonprestressed tensile steel
T_{ps}	=	tension force in prestressing steel

Strains

ε_{cu}	=	strain in concrete top fiber at ultimate

ε_{ps} = strain in prestressing steel at ultimate

$\Delta\varepsilon_p$ = increase in prestressed tendon strain

$\Delta_{\varepsilon p(ext)}$ = change in the external prestressed cable strain

Dimensionless Factors

δ = opening of the crack

Δ = deflection at midspan

ϕ_c = resistance factor for concrete

ϕ_s = resistance factor for reinforcing steel

ϕ_p = resistance factor for prestressing steel

PPR = partial prestressing ratio

S/d_p = span-to-depth ratio

ω = combined reinforcing index

ω_s = reinforcing index of nonprestressed tensile steel

ω_{ps} = reinforcing index of prestressing steel

ω_s' = reinforcing index of nonprestressed compressive steel

ρ_s = reinforcement ratio of nonprestressed tensile steel

ρ_s' = reinforcement ratio of nonprestressed compressive steel

ρ_{ps} = reinforcement ratio of prestressing steel

β_1 = concrete compression block reduction factor

Chapter 1

Introduction

1.1 General

A large number of bridges are in a deteriorated stage because of their exposure to severe weather conditions, aging, continuous increase in the traffic volume, poor and/or inadequate maintenance, insufficient protection of the prestressing steel, corrosion of steel reinforcement resulting from exposure to highly humid climates and salt water environment, temperature effects and change in design specifications. Deterioration due to environmental changes is the most significant and growing problem. Structures can be damaged due to blast loading, fire, earthquakes, overloading, etc. Unsatisfactory performance with respect to serviceability and ultimate strength can also be attributed to several factors such as errors in design calculations, improper concrete mix, large deformations and cracking due to temperature effects and time-dependent effects of creep and shrinkage of concrete and relaxation of prestressed steel. Originally a large number of existing bridges were designed to carry lighter vehicle loads and lower traffic volumes at slower speeds than what is common today.

The key to improving highway bridges in the 21st century will be the use of advanced or enhanced materials, inspection technology, design procedures, construction methods, operational practices, maintenance and rehabilitation technology, and management techniques. Technology that permits rapid repair and return to service with minimal disruptions to safety and traffic flow has become essential nowadays. Rehabilitation and strengthening have become a major focal point for the nation's aging bridges.

At present there is widespread deterioration of bridges in North America, Europe, and Japan, resulting from a steady increase in weight of highway vehicles (as high as forty percent over the original design loads), the application of the de-icing salts, etc. as described previously. In addition, the expected retirement dates of concrete bridges built prior to 1950 will be reached by the year 2000, which will present a staggering economic burden (Klaiber et al. 1987).

The eighth annual report of the Secretary of Transportation to the Congress of the USA reported that nearly 40% of almost 600,000 inventoried highway bridges located on public roads in the United States were built before 1940. Many of these bridges which have not been adequately maintained are classified according to the Federal Highway Administration's (FHWA 1986) criteria, as either structurally or functionally deficient and are in need of rehabilitation or replacement. More than 100,000 of these bridges are judged to be structurally deficient because of deterioration or distress or just being not capable of carrying the minimum acceptable load. Another 100,000 are considered functionally deficient because of being old or inadequate for current requirements (Nowak et al. 1992).

The 1993 report of the Secretary of Transportation to the U.S. Congress on the status of the nation's bridges concludes that, although bridge conditions are improving slightly, approximately 35% of the bridges in the U.S. are classified as either structurally deficient (21%) or functionally obsolete (14%) (FHWA, 1993).

Although no similar data exists presently for Canada, it may be assumed that roughly 60,000 to 100,000 bridges exist in Canada. In Quebec, half of the maintenance budget of the Ministry of Transportation is spent on concrete structures damaged by corrosion of steel (Chaallal, 1991). Within Europe, the annual cost of corrosion has been estimated at 1000 million pounds per year (Clarke, 1993).

Structural deficiency does not necessarily imply that a bridge is unsafe. It does, however, mean that a structure is unable to carry the vehicle loads or tolerate the speeds that would normally be expected for that particular bridge in its designed system. Functional obsolescence means that the bridge has inadequate width or vertical clearance for its associated highway system. In some cases, bridges become functionally obsolete because of highway improvements on the approaches to the bridge, such as lane additions or widening of approaching roads. In other cases, a bridge may be classified as functionally obsolete through a redefinition of desired standards.

The disposition of deficient bridges involves decisions ranging from closing to traffic for replacement or repair, through posting for restricted use. Each decision involves clear economical and safety implications. Efficient evaluation methods as well as repair and strengthening techniques are required.

In addition to maintenance, strengthening is necessary for old bridges to improve their load carrying capacities for today's increased traffic and bring them up to the current standards. Strengthening is preferred over demolishing and replacement if it can be the best economic solution.

The current estimate of bridge replacement and rehabilitation program is about 50 billion dollars (US), and to repair all the United States's concrete structures it will cost 1 to 3 trillion dollars (US) (Fickelson, 1990). Recent estimates put the rehabilitation bill to repair only existing Canadian parking structures at 4 to 6 billion dollars (CAN) (Bédard, 1992). The high cost of replacement can be avoided by rehabilitating bridges at earlier stages before the deterioration reaches an advanced stage. Because of its cost effectiveness for many cases, the FHWA (1993) recommends that rehabilitation be given primary consideration before replacing a bridge.

1.2 Background of Problem

The present state of deterioration in existing bridges is due to the use of two fabrication practices in early fifties:

1. Addition of calcium chloride to the concrete mix as a plastisizer to accelerate curing of concrete in cold weather. In the presence of water and air, calcium chloride causes corrosion of steel bars and the prestressing wires and cables embedded in the concrete. This corrosion causes pitting of the prestressing strands and loss of cross-sectional area of steel bars. Corrosion produces bursting forces on the surrounding concrete leading to cracking and spalling. External cables with insufficient mortar protection are also exposed to corrosion and rupture. Corrosion of the steel leads to light and sometimes severe rust formation. Failure of a structure caused by disintegration of concrete due to the effects of aggressive water, chemical attack and bursting pressures, could be generated by corroding reinforcement.
2. Concrete covers to the prestressing strands were not sufficient. Adequate cover of high quality concrete normally protects the embedded steel from corroding by preventing passage of water because of leakage through joints.

1.3 Preservation Techniques of Concrete Structures

Bridge structures, like any other structure, deteriorate with time. Deterioration can be attributed to inadequacy of design, detailing, construction and quality of maintenance, and also to overloading, chemical attacks, atmospheric effects, abnormal floods and erosion, abnormal earthquakes, etc. Parking structures have been very often compared to bridges because of the traffic environment they serve in, and also due to the fact that they are exposed to changes in weather conditions. It is therefore necessary to understand what is meant by the following terms: *Maintenance, Repair, Rehabilitation, Reconstruction, Strengthening,* and *Retrofitting* of existing bridges. These alternatives to *replacement* are all interrelated and are sometimes difficult to separate.

Maintenance refers to the work needed to be done to preserve the intended load carrying capacity of the bridge and safety of the public using it, and to control potential future deterioration.

Repair is defined as the process of restoring the original strength condition to a component of the structure after its loss from accidental or fatigue damage, fire, corrosion, deterioration, overloading, etc. It is the technical aspect of rehabilitation.

Rehabilitation is defined as the overall or comprehensive process of restoring the load-carrying capacity and serviceability of a structure caused by deterioration with time, to its original level. In other words, the purpose of rehabilitation of deteriorated structures is to re-establish their capability to accept, with the sufficient safety and desired durability, all the effects and to respond to all the functional requirements which have been anticipated by the original design. Rehabilitation often involves both repair and strengthening.

Reconstruction has the purpose of making an existing structure, which is not necessarily deteriorated, capable to accept higher effects and to respond to higher functional requirements compared to those anticipated by the original design.

Replacement refers to reconstruction of the whole structure or of its major components. Replacement is often implemented when the cost and/or the extent of repair or strengthening is beyond the acceptable economic or technical limits.

Strengthening of existing structures, is the process of upgrading their load carrying capacity of a bridge. Strengthening can be realized by decreasing the load effects (e.g., by decreasing the dead load, decreasing the span, or changing the structural system), by increasing the resistance of the members (e.g., increasing the cross section, or by adding reinforcement, or steel plates) and by changing the state of stress (e.g., additional prestressing). Strengthening can be a cost effective alternative to complete replacement.

Retrofitting of concrete structures is an improvement intended to restore a specific structural function, upgrade a bridge status to a higher performance level, or modify the structural characteristics to make a bridge structure fit an additional capacity requirements. Retrofitting may include repairs of damaged structures to restore structural performance with respect to service and ultimate load levels, strengthening of undamaged structures to improve structural behaviour, replacement, etc.

The term *durability* is used to indicate the period of time over which a material performs its intended function in a given environment. There are several ways to improve durability of a new structure. For examples, the concrete cover should be of the appropriate thickness and a concrete with a low water to cement ratio should be utilized. In addition, epoxy coated bars can be used and a cathodic protection system can be installed. All these measures can be expected to improve durability.

Several different techniques have been successfully developed for the repair and strengthening of a given structure or a part of a structure, depending on the type of construction and the given situation. A review of these techniques is given by Klaiber et al. (1987). Two of these techniques have been used most frequently in strengthening and rehabilitation of concrete or composite structures, particularly bridges, comprise addition of external reinforcement in the form of reinforcing steel cables or plates as described below:

a) Strengthening by External Cables: in this technique, high strength steel cables are anchored to steel or concrete girders and post-tensioned to improve the elastic response, the ultimate capacity and the fracture behaviour. Use of external prestressing has the advantage of reducing the dead load of the structure due to reduction of the web thickness of the cross-section, and provides saving in the construction work. Also, old or damaged cables can be easily replaced and the losses in prestressing are smaller than in internal bonded tendons. Because of the many advantages of external prestressing, it has become a primary technique for the rehabilitation of existing old structures and is being increasingly considered in the construction of new structures. External tendons are primarily designed to ensure horizontal prestressing of the structure and generally represent only a portion of its total flexural reinforcement. The remaining reinforcement may consist of ordinary mild reinforcing steel, internal prestressing steel or a combination of both, depending on the structural system and the type of construction. Minimum reinforcement is also necessary to control the distribution of cracks and to limit the crack widths during the construction period, in the serviceability limit state due to loads and/or imposed deformations, and in the regions where tensile stresses and possible cracks may occur due to simplified tendon profiles. In the design of new concrete bridges, external tendons constitute the primary reinforcement and, thus, the analysis and design could be achieved using methods similar to unbonded post-tensioned construction. The most important difference between the use of external prestressing tendons in the design of new concrete structures and in strengthening of existing structures is in the level of externally applied load and the extent of cracking and deflections at the time of application of the external prestressing.

Externally prestressed steel tendons, however, need to be protected under service conditions against environmental attacks causing corrosion, and against fire and destruction. In North America, the normal practice is to use cement grouting of the tendons for protection. However, this type of protection precludes from restressing existing tendons or replacing old or damaged tendons. Also, cracking of this grout by shrinkage exposes the tendons very often to corrosion by de-icing salts. Several concrete structures prestressed with external steel tendons have suffered from corrosion problems due to insufficient protection of the tendons from exposure to humid climates and salt water environment.

b) Strengthening by Bonded Steel Plates: in this technique, steel plates are bonded using epoxy adhesives to the soffits or the sides of concrete members which are under tension and show signs of cracking. This technique has been a frequent practice for strengthenig of flexural members such as beams and slabs for more than twenty five years. It has also been used to increase the live load capacity of existing bridges. However, a significant loss in the bond strength was observed when the steel plates were subjected to a simulated rain environment. Corrosion developing at the adhesive-steel-concrete interface when the structure is exposed to aggressive environments is a major disadvantage of this strengthening technique. Such corrosion adversely affects the bond between the plates and the concrete and makes the structure susceptible to overloading and possible major distress. In addition, since steel is a highly magnetic material and a good conductor of electricity, the application of bonded steel plates is very limited in sensitive structures which are subjected to electro magnetic fields.

One of the most promising avenues to overcome the corrosion problems of steel and the associated durability problems in reinforced and prestressed concrete structures is the use of non-corrosive reinforcement instead of steel. Advanced composite materials such as Glass Fiber Reinforced Plastic (GFRP) and Carbon Fiber Reinforced Plastic (CFRP) have superior corrosion resistance and offer an excellent solution to corrosion problems. Recently, there has been a tremendous amount of research forwards the use of Fiber Reinforced Plastics (FRPs) in the form of reinforcing bars, prestressed cables, plates or fabric sheets replacing conventional steel reinforcement in the construction of new structures or rehabilitation and strengthening of existing old structures. In addition to their excellent non-corrosive characteristics, FRP reinforcement has high strength-to-weight ratio that can provide high prestressing forces without additional weight to the structure. It also has good fatigue properties (CFRP has fatigue strength three times higher than steel) and low relaxation losses which can increase the service life and the load carrying capacity of concrete structures. FRPs also have excellent electromagnetic resistance which makes them usable in sensitive structures as opposed to steel.

The present investigation is another effort to establish the efficiency of FRP reinforcement in strengthening and enhancing the performance of concrete structures and to provide further understanding of the behaviour of structural concrete members strengthened with this type of materials.

1.4 Objectives and Scope

The prime objective of this research is to examine the effectiveness of fiber reinforced plastics when used to strengthen and retrofit reinforced concrete and partially prestressed concrete members. To achieve this objective, two experimental projects were carried out in the structural laboratory at Concordia University. The first project is concerned with strengthening of concrete members using externally prestressed carbon fiber reinforced plastic cables. The second project investigates retrofitting of concrete members with epoxy-bonded composite straps.

The main objectives of the first project are:

1. To study the behaviour of partially prestressed concrete members after being strengthened with externally post-tensioned FRP ropes, particularly those made of carbon fiber reinforced plastics, in lieu of steel cables. The CFRP ropes used in this investigation are those manufactured by Tokyo Rope, Japan, and known as Carbon Fiber Composite Cables (CFCC).
2. To determine the increase in ultimate moment capacity of reinforced and partially prestressed concrete members strengthened using external CFCC cables.
3. To investigate the effects of important parameters such as the span-to-depth ratio (S/d_p) and the level of internal prestressing expressed in terms of the partial prestressing ratio (*PPR*) or the reinforcing index (ω) on the strength and behaviour of reinforced and partially prestressed concrete members after strengthening with the CFCC cables.
4. To evaluate the stress in the external CFCC cables, f_{pf}, at failure to be used for calculation of the ultimate moment capacity of the strengthened members.

The experimental program of this project comprised testing a total of twelve reinforced and partially prestressed concrete simply supported beams, all having a 150×280mm rectangular cross section. During testing, all the beams were subjected to two symmetrical concentrated loads applied at one-third of the span length in small increments. The twelve beams were divided into three groups; each group consisted of four beams of the same length but with different levels of internal prestressing. The span-to-depth ratio varied from one group to another. Three S/d_p ratios were used: 10.7, 17.85, and 25. Within each group, three of the beams had different partial prestressing ratios, $PPR = 0$, 0.53 and 0.7 (or reinforcing index $\omega = 0.05$, 0.142, and 0.234, respectively). Note that $PPR = 0$ indicates a reinforced concrete member. The three beams were subjected to load large enough to produce considerable cracking and deflection and then strengthened with the external CFCC cables before being reloaded up to failure. The external cables used were CFCC 1×7- 5mm and 7.5mm diameter. The fourth beam in each group had a PPR = 0.53 ($\omega = 0.142$) and was used as a control beam loaded monotonically from zero up to failure without strengthening.

Analytical equations were suggested for estimation of the change in stress in the external CFCC cables at any load level after strengthening and at ultimate. The effects of S/d_p and PPR (or ω) on the change in this stress were examined.

The main objectives of the second project are:

1. To study the feasibility of using composite straps in the form of Polyester straps (32mm wide) bonded with epoxy adhesive to the tension side of the member instead of steel plates for retrofitting of prestressed concrete members.

2. To investigate the feasibility of retrofitting precast-prestressed concrete railway crossties, instead of replacing them by new ones, using the epoxy-bonded composite straps placed on the top surface of the ties after being subjected to cracking caused by rebound forces resulting from the wheel impact.

Two full scale precast-prestressed concrete railway crossties with different level of prestressing were loaded from the bottom to produce cracking on the top surface similar to the cracking produced by rebound forces. The ties were then retrofitted by bonding two layers of straps on the top surface using epoxy adhesive and then reloaded up to failure. The synthetic Polyester fabric straps were of type 105 WGSD supplied by Caristrap International Inc. The epoxy adhesive used was a mix of a fast weld epoxy resin and harder No.10 manufactured by Ciba-Geigy Inc. and distributed by MF Composite Inc. The experimental results were compared to those obtained from a previous extensive study conducted by Peter (1992) on the performance of the crossties without strengthening.

1.5 Outline of Thesis

In the following chapter, the composition, material characteristics and mechanical properties of different types of fiber reinforced plastics are discussed. The advantages and disadvantages of this type of this material are reviewed and some of their applications as reinforcement in bridges and other structures are presented. Some of the problems encountered in the design of concrete structures with FRP reinforcement are also discussed.

Chapters 3 and 4 are devoted to the first project on strengthening of concrete members using externally prestressed carbon fiber reinforced plastic cables. In Chapter 3, a review of previous research carried out on the use of FRP reinforcement for prestressing is presented. Particular attention is given to investigations on external prestressing with FRP tendons. A detailed description of the present experimental program on the use of CFCC cables for strengthening of partially prestressed beams is given. The procedure for preparation, fabrication and instrumentation of the test specimens is explained in detail. The material properties of the concrete and steel and the characteristics of the CFCC cables used are given. The test set-up and testing procedure are described in detail. The results of this experimental program are presented and discussed in Chapter 4. Graphs of the load-deflection response, the variation of the strain in concrete, the strain or stress in the reinforcing steel and the change in the stress in the external CFCC cables with the applied load are presented. The ultimate flexural strength and the modes of failure after strengthening are also discussed and compared with those of control beams without strengthening. The details of these experimental results are given in Appendix A in the form of tables and photographs.

The second project on the use of fiber composite straps for retrofitting of precast-prestressed concrete railway crossties is presented in Chapter 5. First, a literature review of the previous work on strengthening of concrete members with bonded composite plates or fabric sheets is given. Then, a detailed description of the experimental program on retrofitting the crossties with the epoxy bonded Polyester straps is given. Also, the experimental results and observations are presented and discussed.

Finally, a summary and conclusions of the two projects presented in this thesis are given in Chapter 6. Recommendations are also given for further research in the area of strengthened and rehabilitation of concrete structures using advanced composite materials.

Chapter 2

Advanced Composite Materials

2.1 General

One of the major problems that are facing structural engineers all over the world is the deterioration of concrete structures due to corrosion of steel reinforcement caused by exposure to aggressive environments. Deterioration due to corrosion reduces the durability and the service life of the structures and results in costly repair and safety hazards. To date, there has been considerable effort made on the development of techniques to avoid corrosion and to improve the durability of concrete structures. Some of these techniques include replacement of the steel reinforcement with epoxy coated, galvanized or stainless steel bars; cathodic protection of the reinforcement; grouting of prestressing tendons with cementitious grout to provide alkali protection to the steel; and treatment of the concrete surface with silanes or siloxanes. However, none of these existing methods has proven to be fully effective in providing perfect protection for steel. Consequently, it appears that the only effective method to avoid corrosion is to remove the steel reinforcement from the concrete and replace it by non-corrosive materials. Recently, it has been proposed to use fiber reinforced plastic (FRP) reinforcement as a substitute for conventional steel reinforcement. So far, the use of this new type of reinforcement has shown a promising solution to durability problems caused by corrosion of steel.

The main attributes and advantages of fiber composites include high tensile strength, high stiffness, high corrosion resistance, light weight, low mechanical relaxation, good fatigue strength, non magnetic and non-conductive properties. Fiber reinforced plastics are not new materials, though definitely younger than prestressing steel, and were not at all developed for structural engineering purposes. The development of these materials essentially began more than four decades ago (in the early 1940's), for military, mechanical and aerospace applications such as rockets and satellites, wing skins for aircrafts, and even main rotor blades for helicopters and automobiles where high strength and light weight are of paramount importance. The introduction of these materials to civil engineers is, however, relatively new. They can be used in the form of discontinuous fibers, as grids, ropes, or typically in the form of rods made of high performance fibers impregnated in a polymeric matrix of plastic or epoxy resin to create composite action and exhibit physico-mechanical properties that are desirable for use as structural reinforcement for concrete, and satisfy the requirement for non-metallic prestressing tendons.

In this chapter, the composition, material characteristics and mechanical properties of different types of FRP reinforcement are discussed. The advantages and disadvantages of this type of material are reviewed and some of their applications as reinforcement in bridges and other structures are presented. Some of the problems encountered in the design of concrete structures with FRP reinforcement are also discussed.

2.2 Composition and Properties of Fiber Reinforced Plastic

A fibrous composite is obtained when fibers of relatively high strength are embedded or impregnated in a light matrix resin. The fibers in this manner improve the stiffness and strength characteristics of the resin. Some of the factors that influence the mechanical characteristics such as strength and stiffness of a fiber composite product are:

- the mechanical properties of the fiber and matrix,
- the fiber volume fraction of the composite,
- the degree of fiber matrix interfacial adhesion,
- the fiber cross section,
- the fiber orientation within the matrix,
- the method of manufacturing.

Experiments have shown that the axial tensile strength of an FRP element depends on the fiber strength and on the fiber volume fraction of the composite element. The fibers are very sensitive to transverse pressure. Hence they are parallely embedded in the polymeric matrix resin. The desirable functional requirements of the fibers in a fiber reinforced (fiber/matrix) composite are:

- high modulus of elasticity (to provide efficient reinforcement and stiffness to the composite),
- high ultimate strength,
- low variation of strength between individual fibers,
- stability of strength during handling and fabrication,
- uniformity of diameter and surface conditions.

The typical commercially available non-metallic fibers generally considered for FRP reinforcement: Glass (inorganic fibers), Aramid, and Carbon (organic fibers) (Rostásy, 1993). The non-metallic fibers may be characterized by their strength, total strain at failure, durability, fatigue resistance, and by their composite behavior. Other factors to consider in the selection of fibers are electrical conductivity, coefficient of thermal expansion, upper use temperature, resistance to radiation, and cost. The synthetic fibers have diameters of only a few thousandths of an inch (Dolan 1990). The small size allows the fibers to develop very high tensile capacities. A common characteristic of the synthetic fibers is that their behavior is nearly linear up to failure.

2.2.1 Properties of Fibers

The strength of FRP reinforcement varies depending on the type of fiber and their content in the matrix. The term fiber is used in its conventional sense to describe the continuous fibrous phase of the composite material. It should be noted, however, that the toughness composite material improves by using short fibers rather than continuous fibers. The stress-strain relationship for all FRPs is linear up to failure. Distinct yield point is not observed and the FRPs may reach failure while in the elastic condition. The tensile strength of FRPs is much higher than the tensile strength of steel. This could be considered as an advantage for design of high strength slender structural members. However, in combination with low modulus of elasticity in comparison to steel, and small ultimate strain in tension, design of FRP reinforced members to satisfy serviceability requirements for deflections may be troublesome. Another problem arises from the fact that the coefficient of thermal expansion for FRPs varies from negative values to $9\text{-}10\times10^{-6}$ m/m/°C (Svecová, 1994). In ordinary reinforced concrete both steel and concrete have very similar values of coefficient of thermal expansion. The difference in thermal properties of FRP and concrete may lead to loss of bond and additional thermal stresses within the structure. The tensile strength of FRPs and their modulus of elasticity were found to be affected by high temperatures. According to Japan Society of Civil Engineers (JSCE, 1993), if temperatures exceed 60 °C, every additional 10 °C increase in temperature will induce a decrease in tensile strength of 50 to 100 MPa, and in Young's modulus of 1 to 2 GPa. Therefore, in design, an adequate concrete cover for FRP rebars has to be provided, or higher safety factors for the tensile strength must be determined.

The fibers are almost ideally elastic and brittle in axial tension. The axial tensile strength and the axial Young's modulus depend on the properties of the components. For common tensile elements, the transverse modulus of elasticity, ranges between 1/3 to 1/2 of the axial modulus.

The basic function of fibers in composite materials is to carry the load along the direction of reinforcements. The desirable mechanical characteristics of theses fibers are: high tensile strength, high elastic modulus, low creep and relaxation, and stability at normal ambient temperature. The fibers often have relatively low shear strength. While steel has a shear capacity equal to about 1/3 of its tensile capacity, some fibers have shear strength only 5% to 10% of the tensile strength (Dolan, 1990). FRP reinforcement is inherently brittle because high modulus fibers exhibit a nearly linear stress-strain relationship up to their ultimate tensile capacity. Maximum strength is achieved by orienting the fibers in the longitudinal direction of the member. Tensile capacity of an FRP rod is based almost exclusively on the fiber content since the resin strength is of an order of magnitude lower than that of the fibers.

2.2.2 The Matrix

The term Matrix is used to describe the combination of materials that bind the fibrous phases together. The matrix, generally has poor mechanical characteristics. This phase includes the resin and the filler constituents as well as the secondary constituents (i.e., additives such as flame retarders, UV absorbers, stabilizers, viscosity modifiers, catalysts, accelerators, mold release agents, coupling agents, etc.) which are typically mixed into the neat resin materials during processing of the composite material. The matrix with its low strength does not significantly contribute to the strength of the composite. The strength and the modulus of elasticity of an FRP element mainly depend on the strength of the fibers and on their volume ratio which is usually in the range of 45 to 70 volume percent (Rostásy, 1993); the remainder is resin. Tendons manufactured from fibers encased in a resin usually have the stresses reported on the basis of the gross section. Therefore, the fiber stresses are 1.7 to 1.4 times the average tendon stress (Dolan, 1990). The primary and basic functions of the resin matrix in FRPs are:

a) to provide lateral support to the fibers,
b) to hold and bind the fibers together as a single unit so as to prevent shear between them,
c) to keep the fibers oriented in the required directions,
d) to protect sensitive fibers from surface damage and local transverse stresses during handling and fabrication and over the service life of the composite,
e) to protect against fatigue abrasion and corrosion attack,
f) to distribute the fibers and to separate them so that any catastrophic propagation of cracks is avoided,

g) to equalize the forces of the non-ideally parallel fibers among the fibers by interlaminar shear transfer within the compound body,

h) to transfer stresses to the fibers efficiently by adhesion (bond) and/or friction,

i) to transfer the force of broken fibers to unbroken ones by interlaminar shear, and to give them sufficient dimensional stability over the required range of service temperature,

j) more importantly, the matrix distributes an applied load and acts as stress-transferrer so that when an individual fiber fails, the composite does not lose its load carrying capability. Durability, interlaminar toughness, shear, compressive and transverse strength are also provided by the matrix resin.

A desired characteristic of most advanced composites is their light weight and their high strength-to-weight ratios. This is attained by using matrices with much lower density than the fibers. Matrices can be organic, metallic or ceramic; the most common are resinous materials such as polymers. Durability of FRPs is controlled by the durability of resins in which the fibers are embedded. One of the reasons of deterioration of resins is water saturation. Some of the important material characteristics to be considered in selecting a matrix for a structural FRP are: stiffness, strength, fracture toughness, thermal and electrical conductivity, upper use temperature, coefficient of thermal expansion, processing temperature, chemical shrinkage during processing, ability to impregnate and bond to fibers, flame resistance, sensitivity to environmental factors such as moisture and chemicals, and ultraviolet radiation. Perhaps the most important consideration for commercial applications is cost.

2.3 Types, Characteristics and Classification of FRP Reinforcing Products

As mentioned earlier FRPs are composite plastic materials consisting of two components: a high strength, high stiffness reinforcing fiber material and a matrix/resin. There are different types of FRPs reinforcement, but the most commonly used ones are carbon (CFRP), aramid (AFRP), and glass (GFRP). The final form of these composites can be in the shape of rods, grids, gratings, sheets, tapes, fabrics, strands, cables, tendons, ropes, and structural shapes (folded plates, singly or doubly curved shells). Rods and cables are used for prestressing applications, reinforcement for concrete structural members, or for ground anchors. Tunnel lining and reinforcement for slabs, walls and panels may be produced using two or three dimensional grids. Gratings are actually manufactured for purposes other than reinforcement. They are mainly for construction of walkways, handrails, and raised floor systems. Sheets and small diameter filaments can be used for the repair of existing structural systems.

Fiber reinforced plastics can be classified by material or by shape. The classification according to Japan Society of Civil Engineers (JSCE, 1993) can be seen in the charts given in Figures 2.1 and 2.2. They have shapes of regular reinforcing bars, or rectangular cross section. Two and three-dimensional meshes are available, and also other different shapes, as for example sheets, and grids.

FRP are anisotropic materials, most important for the practical application is their axial tensile strength which is the strength parallel to the fibers (Rostásy, and Budelmann, 1993).

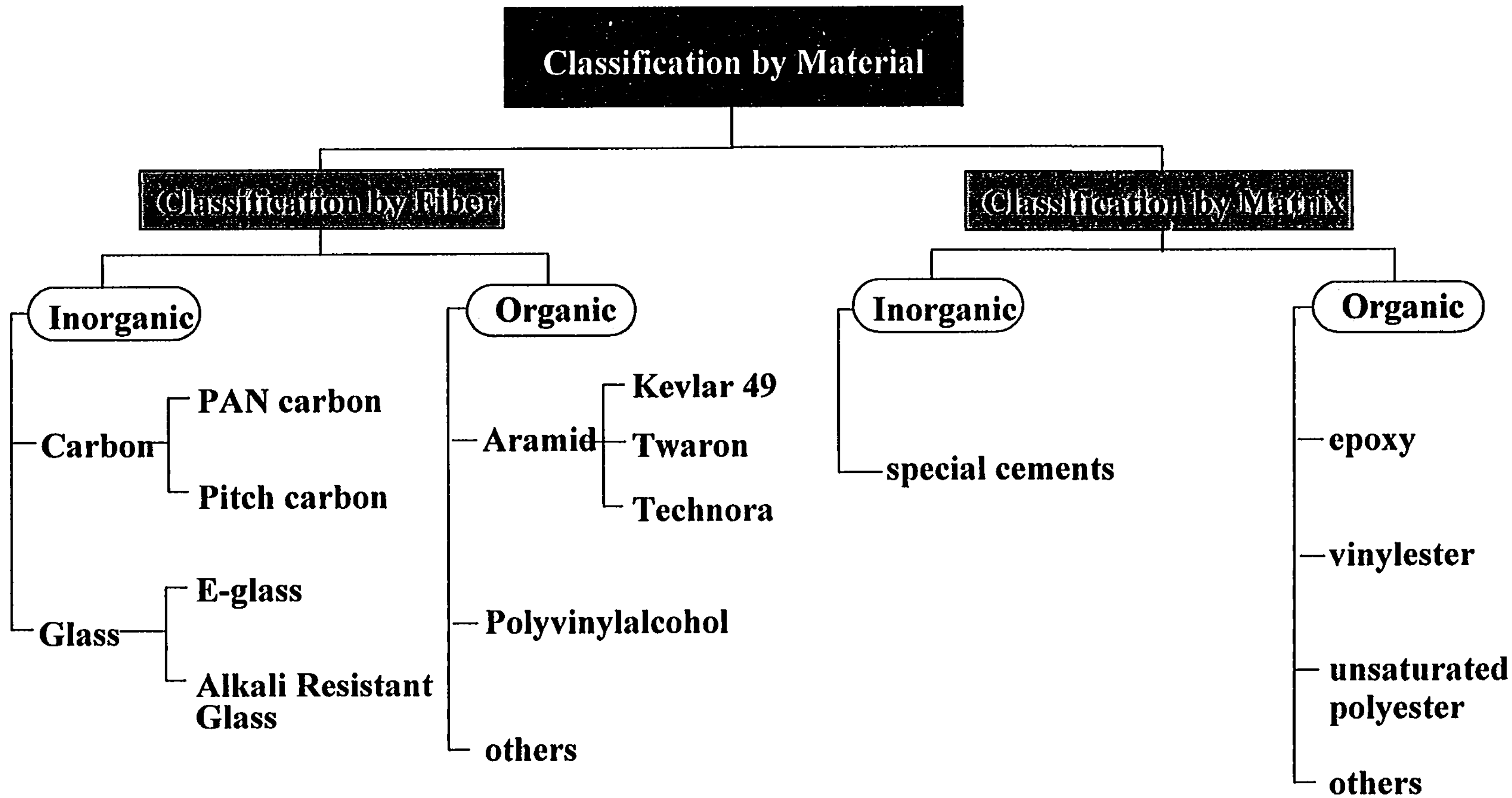


Figure 2.1 Classification of FRP Reinforcement by Material

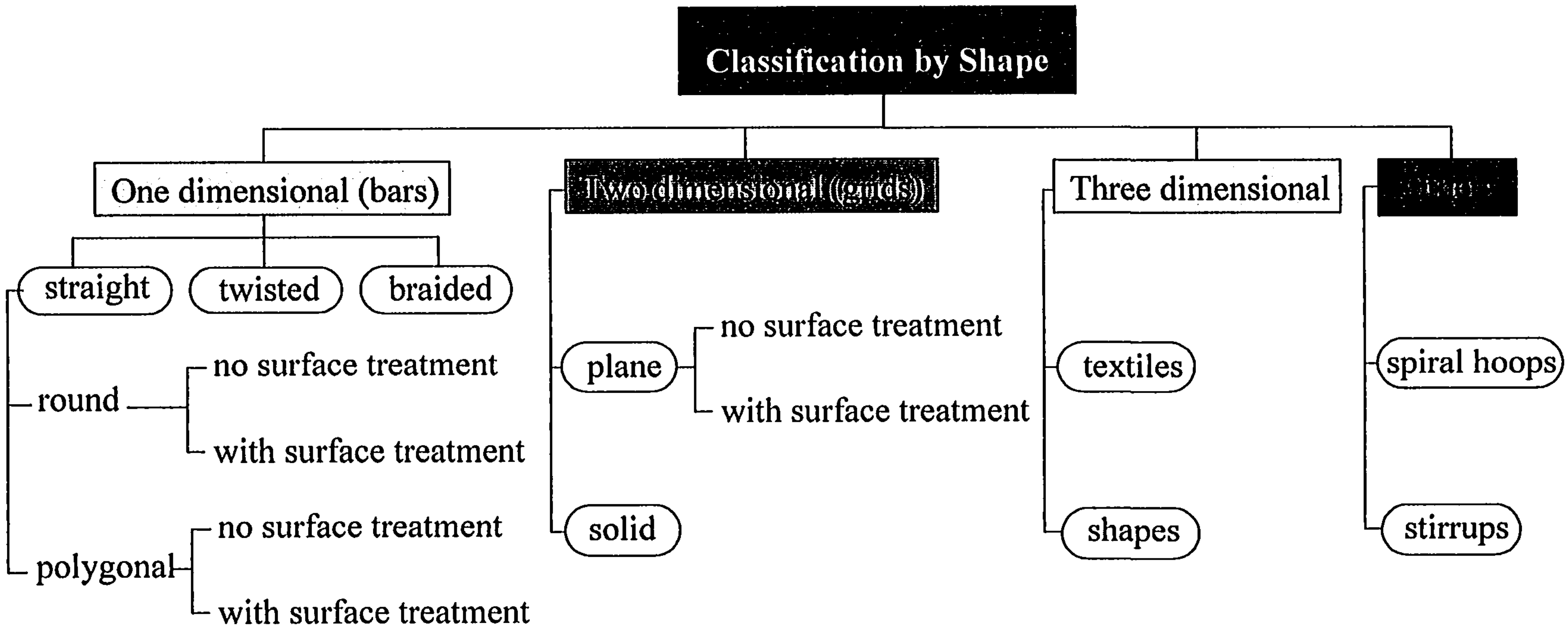


Figure 2.2 Classification of FRP Reinforcement by Shape

Some of the available FRP reinforcing products are described in the following subsections.

2.3.1 Carbon Fiber Reinforced Plastics (CFRP)

Carbon fiber reinforced plastic is a composite of synthetic fibers, similar to those used for making bullet-proof textiles, or pitch fibers, and a durable resin. CFRP reinforcement can be made in the form of rods, tendons and ropes. They are characterized by:

- having a high tensile strength. The tensile strength of typical CFRP tendons and wires is approximately 1700 GPa (246.6 ksi), with maximum strengths around 3300 MPa (478.6 ksi),
- having the highest tensile modulus of elasticity (typically 65 percent or more of the modulus of elasticity of steel). The elastic modulus of typical CFRP tendons varies from 140 to 165 GPa (20.3 to 23.9 ksi),
- having the least tensile strain at ultimate. The ultimate strain lies between 1.2% and 2%,
- having low relaxation. According to Rostásy (1988), the loss of relaxation for carbon fiber is negligible when the initial stress is equal to 50% of the ultimate stress,
- good fatigue properties,
- having axial coefficient of thermal expansion equal to 0.2×10^{-6} m/m/°C (0.11×10^{-6} in/in/°F),
- being the least sensitive FRP material to any kind of environmental influence.

Most of CFRP tendons have a tensile strength of about 3 GPa (435 ksi) and a modulus of elasticity of 230 GPa (33.36 ksi).

According to Hollaway (1989) two types of carbon fibers are available: (1) synthetic fibers known as polyacrylonitrile (PAN) which are similar to fibers used for making textiles, and (2) pitch-based carbon fibers, obtained by the destructive distillation of coal. The manufacturing process of carbon fiber consists of carbonization, which is preheating PAN fiber under tension in air 250 °C in order to avoid shrinkage, and gain strength. This process also changes its color from white to yellow, to brown, and finally to black. The stiffness of the carbon filament can be increased further by heating it up to 2200 °C (Hollaway, 1978).

The two types of carbon fibers are discussed in details as follows:

a) Polyacrylonitrile CFRP: used to make unidirectional Carbon Fiber Composite Cables (CFCC) developed by Tokyo Rope Manufacturing Company Limited. (more information is presented in Chapter 3). CFCC fails in a brittle manner without showing a yield point and a yield plateau like steel. It has no plasticity regions in the stress-strain relationships. For this reason, when compared to steel strands, CFCC have vastly superior anti-fatigue properties when subjected to repeated tensioning. Although CFCC have some excellent properties (ultimate tensile strength 1.8 GPa). They also have some mechanical deficiencies such as a low Young's modulus (137 GPa) and small elongation at failure (1.6%) as compared to steel. Table 2.1 shows a comparison of the mechanical characteristics of 1×7 strands CFCC 12.5 mm diameter which is commonly used in concrete construction, and the equivalent cable made of 1×7-12.5 mm steel strands.

Figure 2.3 shows a comparison between the tensile load of CFCC 1×7-12.5mm and the corresponding steel cable with 12.5mm diameter. The CFCC has higher material strength, but since the protective layer around each single wire reduces the effective cross section, it has 11% lower breaking load. On the other hand, the same weight of CFCC carries about four times the load carried by steel. It is also worth to point out the difference in tensile modulus between CFCC and steel. CFCC has a lower modulus (137 GPa) in comparison to steel (200 GPa). This is considered to be an advantage for CFCC since smaller losses of pretension are experienced when the concrete shrinks and creeps.

Zoch et al. 1991 performed relaxation tests on CFCC and steel under equivalent conditions. Cables of both materials with 12.5mm diameter were prestressed with 80% of their proof load and relative decrease in load over time was measured. The results showed that over a period of 100 hours the stress was reduced by 2% for CFCC whereas steel relaxed approximately 8% from the original load. This indicates a superior resistance of Carbon Fiber Composite Cable to permanent stress. Santoh et al. 1993 conducted similar relaxation tests. His tests indicated that at $0.5P_u$, the relaxation for CFCC after 100 hours was 0.48% and that for steel was 1.02%. At $0.65P_u$, relaxation for CFCC was 0.81% while for steel, it was 2.28%. At $0.8P_u$, it was 0.96% for CFCC and 7.35% for steel. This indicates that the relaxation of CFCC is half or less than that of steel.

Table 2.1: Comparison between Mechanical Characteristics of CFCC and Steel Strands (after Santoh 1993, Mitsubishi 1992, and Tokyo Rope 1993)

	CFCC	Steel Strands
Construction	1×7	1×7
Diameter (mm)	12.5	12.5
Sectional Area (mm^2)	76	93
Tensile Load (kN)	> 137	> 160
Yield Load (kN)	-----	> 136
Elastic Modulus (GPa)	137	200
Breaking Load (kN)	161	183
Tensile Strength (MPa)	1760	1850
Strain at Failure (%)	1.6	≈ 6.4
Weight (g/m)	151	774

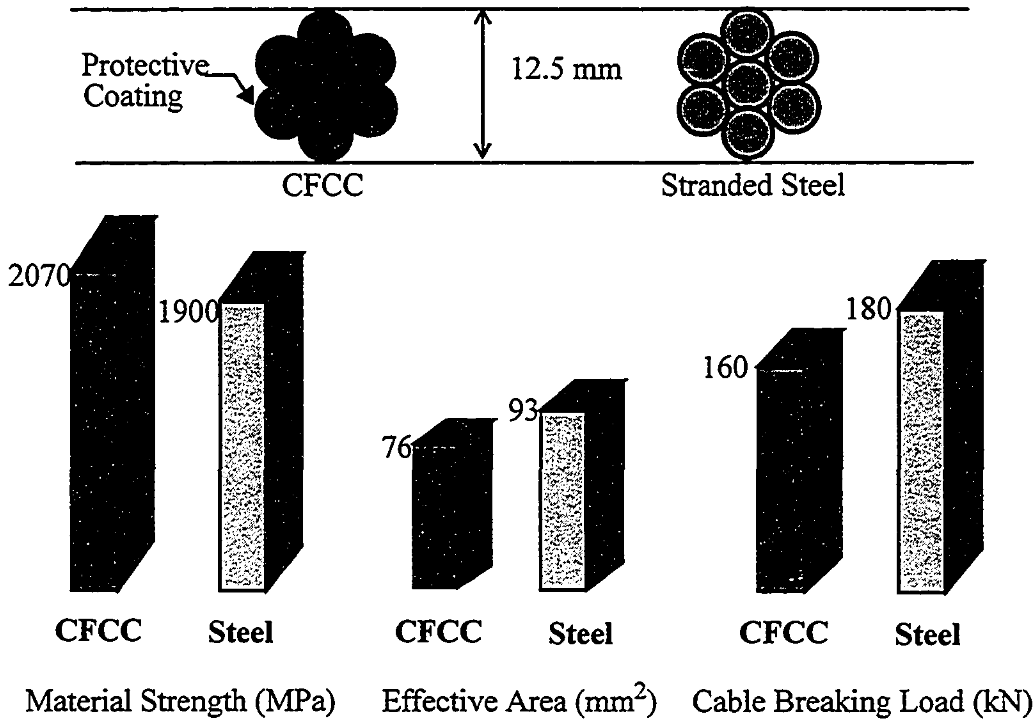


Figure 2.3 Properties Comparison between 12.5 mm CFCC and 12.5 mm Stranded Steel Cable (after Zoch et al., 1991)

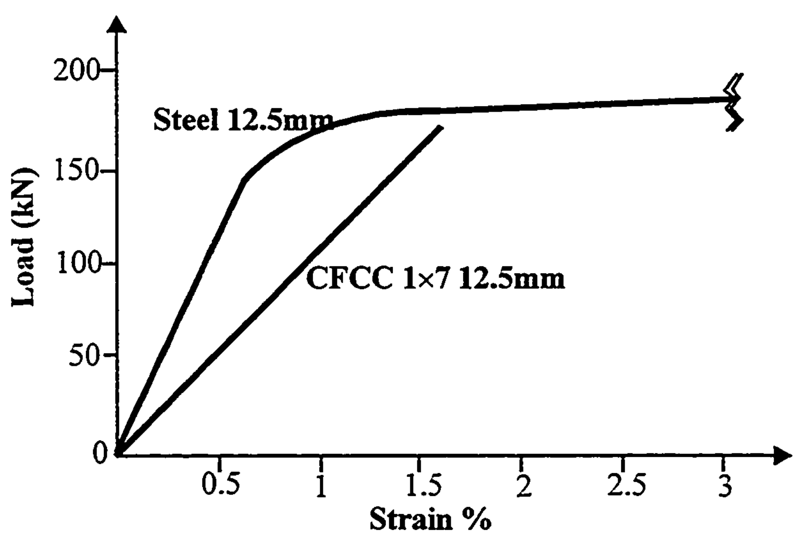


Figure 2.4 Load vs. Strain Curves for 12.5mm CFCC and 12.5mm Steel Cables (after Zoch et al., 1991)

With regard to the load-strain behaviour, Figure 2.4 shows the load-strain diagrams for both CFCC 1×7-12.5 mm and 12.5 mm steel strand. The steel strand exhibits extensive plastic deformation before failure, while the CFCC strain increases linearly with increasing the load until the ultimate fracture load is reached.

b) Pitch-based CFRP: Mitsubishi Kasei Chemical Company of Japan, manufacture round and deformed Leadline CFRP using Dialead coal tar pitch-based continuous carbon fiber and an epoxy resin (Mitsubishi Kasei, 1992). Plain round bar diameters range from 1 to 17 mm (0.04 to 0.67 in) and deformed bar diameters from 5 to 17 mm (0.2 to 0.67 in). The mechanical properties of these rods are: a tensile strength of 1.813 GPa (263 ksi), a modulus of elasticity of 147 GPa (21,000 ksi), and an elongation at failure of 1.3%. Among the good qualities of these rods are: light weight (one fifth the weight of steel tendons (Razaqpur et al., 1993)), high corrosion resistance, good durability, and non-magnetic properties. Wedge-type anchors, as well as grout-type anchors, have been developed for single bar, three-bar, and eight-bar configurations. Leadline CFRP bars are suitable for post-tensioning, in marine floating structures, and for ground anchors for a stress-ribbon footbridge.

The use of CFRP as prestressing reinforcement of concrete structures has increased rapidly for the last ten years. The non-corrosive characteristics of CFRP reinforcement significantly increase the service life of the structures. Although CFRP prestressing reinforcement has excellent properties, the low strain at failure and low elastic modulus in comparison to conventional steel could significantly affect the structural behaviour and ductility of structural concrete members.

CFRP have been proposed for use in long span cable stayed bridges. The reason of this is that to achieve similar axial stiffness as steel, high modulus carbon fiber reinforced plastics are required. Nevertheless, it must be pointed out that, the high specific strength and modulus are less advantageous for the vibrational characteristics of the carbon fiber cables. The latter have quite a negligible damping compared to the steel cables, especially, when the sheathing of the steel cables is grouted with cement mortar aiming at a better corrosion protection.

2.3.2 Aramid Fiber Reinforced Plastics (AFRP)

The term aramid comes from its chemical bases as aromatic polyamide. Aramid fibers have lower weight than carbon fibers, they have toughness similar to glass fibers and a lower Young's modulus than carbon fibers. This means that aramid fibers are superior to carbon fibers in terms of toughness and impact resistance. In addition, the magnetic permeability of aramid fibers is the smallest among these reinforcements. The Young's modulus is ¼ that of prestressing steel, and the specific density is 1/6 that of the prestressing steel. AFRP include the Kevlar® fibers manufactured by DuPont, Twaron® fibers made by Enka in Holland and by Dolan in the United States, and Technora® made by Sumitomo Construction and Teijin Corporation in Japan. Technora does not belong to the aromatic polyamide group, it is aromatic polytheramide fibre. Kevlar® 49 fibers have a linear stress-strain behaviour, but when under bending, they exhibit a high degree of yielding on the compression side (Mallick, 1988). Kevlar® 49 composites have superior performance among other fibers' composites. Although Kevlar® 49 is affected by ultraviolet radiation and strong alkaline solutions, the thermoplastic sheath of the ropes is a secure protection from these sources of attack. Aramid fibers can be used by themselves, or combined with glass, or carbon fibers in a hybrid form. This types of fiber has a fire resistance up to 450 °C (JSCE,1993), has good fatigue, and chemical resistance. AFRP are less sensitive to alkaline solutions than GFRP.

2.3.3 Glass Fiber Reinforced Plastics (GFRP)

Glass fiber reinforced plastic bars are provided with a sanded surface as opposed to steel bars which have ribs. This can be an advantage for GFRP reinforcement because a continuous bond between a bar and the concrete can be developed. Because of the excellent durability properties of GFRP, this reinforcement can be placed close to the surface where it will be most effective. This is in particular the case in aggressive environments where GFRP is an interesting alternative to steel. A location close to the surface is an advantage since it reduces the effect of a lower elastic modulus than that for steel. The basic anchorage problem for GFRP tendons is the low lateral compressive strength. A practical solution, currently used in the industry, is to embed the bars into an epoxy resin cone, or cones, to resist the lateral compressive wedging stresses from a restraining steel sleeve. Because the interlaminar shear strength of the resin matrix is relatively low, the anchorage length for these grouted anchors is significantly longer than the anchorage length for steel prestressing tendons. This may make their use unfeasible for certain designs. Glass fibers have poor resistance to alkali in concrete structures.

2.3.4 Comparison between the Fundamental Characteristics of FRP Reinforcement

Table 2.2 adopted from Mutsuyoshi at al. 1990 gives a comparison between the general fundamental characteristics of fiber reinforced plastics used for reinforcement of concrete structures. The comparison clearly indicate that CFRP reinforcement is the most appropriate among the four types of material as reinforcement for concrete structures.

Table 2.2: Comparison between the Fundamental Characteristics of FRP Reinforcement and Steel (after Mutsuyoshi at al. 1990)

Characteristic	Steel	CFRP	AFRP	GFRP
Tensile Strength	●	●	●	■
Elongation	●	■	■	■
Elastic Modulus	●	■	×	×
Relaxation and Creep	●	●	×	■
Fatigue Strength	●	●	●	●
Alkali Proof	●	●	■	×
Corrosion Resistance	×	●	●	●
Specific Gravity	×	●	●	●
Magnetization	×	●	●	●

●: Excellent, ■: Poor, ×: Bad

2.3.5 Mechanical Properties of FRP Reinforcement

As mentioned earlier, the mechanical properties of fiber reinforced plastic reinforcement depend on the type of fibers, the content of fibers used (which normally varies between 45 and 70), and the manufacturing process. Standard mechanical properties of some FRPs are summarized in Table 2.3 (JSCE 1993). For comparison, the properties of steel and concrete are also included in the table.

2.4 General Advantages of FRP Reinforcement

While steel has the advantage of a long yield plateau and a high modulus of elasticity, FRP reinforcement have other advantages. Depending on the fiber mix, fiber orientation, and fiber content, a wide range of properties can be achieved and the strength and modulus of elasticity can be varied greatly. The main advantages of FRP reinforcement can be summarized as follows:

1. The excellent corrosion resistance makes it ideal for reinforcing concrete structures exposed to high saline or salty environments. Thus, it is not necessary to place the cables within ducts in the concrete, nor it is necessary to provide extensive corrosion protection to external tendons. FRP tendons need no protective coating such as painting, as steel often does.
2. The high tensile strength of FRP can allow for high prestressing forces.
3. The high strength-to-weight ratio of CFRPs results in a specific strength that is 10 to 15 times higher than that of steel. This high specific strength reduces the self weight of the structure. This becomes quite significant with increasing dimensions of the structures.

Table 2.3: Mechanical Properties of Fiber Reinforced Plastics, Steel and Concrete

Type of Fiber		Tensile Strength MPa	Young's Modulus GPa	Ultimate Strain %	Coefficient of Linear Expansion (10^{-6} / °C)
Carbon	PAN	1500 - 2000	120 - 150	1.2 - 1.6	0.6 - 1.00
	Pitch	800 - 1800	80 - 150	0.5 - 1.2	~ 0.7
Aramid	Kevlar 49	1300	66	2.2	-5.71
	Twaron	1360	63	2.2	-2.00
	Technora	1800	54	3.6	-3.00
Glass		750 - 1600	35 - 53	2.0 - 3.2	9.00 - 10.0
Polyvinyl Alcohol		600 - 780	28 - 37	3.0	4.40
Mild Steel		370 - 700	210	12.0	12.00
Concrete		2 - 5	30	0.4	10.80

4. High equivalent modulus. When a load is applied to a cable with a horizontal as well as a vertical span, the elongation consists of the material deformation augmented by a deformation due to the straightening out of the cable. The ratio of the applied load and the observed "strain" (elongation/original distance between the end points) is called the equivalent modulus. This factor is very important in view of the deflection constraints imposed on bridges. A relatively high modulus coupled with a low mass density give CFRP an advantage that increases with the length of the horizontal span and the initial tension.
5. The high fatigue resistance, particularly for carbon and aramid fibers, which can be as much as 3 times higher than that of steel.
6. The excellent electromagnetic resistance which can be an important requirements for facilities requiring no interference in electromagnetic wave transmissions. Typical such facilities will include radar and electronic communications facilities, hospitals and research laboratories with sophisticated electronic equipment.
7. The flexibility of FRP make it easy to assemble and to be formed to any complex shape.
8. The Flexibility of adapting the mechanical properties by selecting different types of fibers and orientation.

In general, FRP reinforcing elements provide durable structures that are free of deterioration caused by corrosion of steel. FRP tendons have the major advantages over steel of being impermeable to effects of sodium chloride. In addition, their light weight makes handling much easier, while the high strength-to-weight ratio of FRP is most significant when spans become very long. FRP composites are also characterized by low (for glass) to high (for carbon) modulus of elasticity in tension and low compression properties.

2.5 Disadvantages of FRP Reinforcement

The advantages of FRP reinforcement listed in the previous section cannot hide the current concerns about two important factors associated with the use of FRP in civil engineering structures, namely, the cost, which is independent of the designer, and the ductility of the structure which must be fully addressed in the design. The following are some disadvantages of FRP reinforcement:

1. The high cost of material in comparison with common civil engineering materials is one of the drawbacks of using FRP particularly for prestressing systems. In comparison with steel, the cost of FRP is 5 to 50 times higher. However, considering the cost of maintenance over the service life of a bridge structure for example to maintain its durability, the use of FRP reinforcement can be considered cost effective.
2. The linear elastic response of FRP renders concrete structures reinforced with FRP not to be ductile. The absence of yield in the FRP behaviour causes brittle failure without warning.
3. The relatively low modulus of elasticity of FRP reinforcement may lead to large deformations under service conditions of concrete structures, particularly those reinforced without prestressing.
4. The ultimate strain at failure, particularly for CFRP, is significantly less than that of steel. In prestressed systems, a portion of the strain capacity is used to prestress the tendons. The remaining strain capacity is available for flexural strain and for safety reserve against failure.
5. The low shear strength of FRP (5 to 10% of the tensile strength) may require longer and larger diameter anchors for prestressing applications.

6. Susceptible to stress rupture effects.
7. Long-term strength of FRP can be lower than short-term static strength.
8. Ultra-violet radiation has been found to damage FRP.
9. Deterioration of Aramid fibers due to water absorption.

Special consideration also need to be given to the effects of low modulus of elasticity of FRPs and the difference in their axial thermal expansion coefficients from that of concrete. A still-unanswered question is the long-term durability of prestressing tendons particularly those made of GFRP. Monitoring systems, such as optical fiber sensors to allow for monitoring the stress-strain behaviour of the full length of prestressing tendon, should provide useful information for their durability in the future.

2.6 Design Considerations for Concrete Structures Reinforced with FRP

Over the past years, significant efforts have been made to introduce FRP as reinforcement in concrete structures. In these efforts, researchers have attempted to address the difficulties that face structural engineers in the design of concrete structures reinforced or prestressed with FRP reinforcement. Design considerations for serviceability, for strength and ductility and for anchorage of prestressing tendons have been addressed. These issues are discussed briefly below.

2.6.1 Design for Serviceability

When FRP reinforcement is used in nonprestressed concrete structures, the design is mainly governed by the serviceability criterion. To ensure enough flexural stiffness for deflection control, higher reinforcement ratios or larger depths are needed. The advantage of the high tensile strength of FRP can therefore be only partly utilized, and reinforcing concrete members with FRP may result in large deformations under service loads particularly after cracking. This makes the use of FRP less attractive in nonprestressed structures but more suitable for prestressed structures.

Moreover, since the moduli of elasticity of FRPs such as AFRP and CFRP, respectively are about 1/4 and 2/3 of that of prestressing steel tendons, the deflection of prestressed concrete beams using FRP rods is larger and the ultimate flexural and shear strengths are smaller than those of concrete beams prestressed with steel tendons. However, the low relaxation losses of the FRP tendons may compensate for the large deformations particularly under time-dependent effects. Methods of computing tendon stresses must specifically account for these conditions.

2.6.2 Design for Strength and Ductility

The strength of a prestressed concrete section is determined by strain compatibility and equilibrium of forces between the compression zone and the prestressing tendons. With metallic tendons, the designer generally assumes that there is sufficient strain capacity to develop the compressive capacity remaining in concrete after prestressing. This assumption is not valid for non-metallic tendons. Analysis of the strength of prestressed structures using non-metallic tendons requires budgeting the strain capacity of the tendon between the flexural requirements and the prestressing requirements. This balance may require a lower initial prestress in the composite tendon in order to allow ductility under overloading conditions.

Since ductility describes the ability of a material, section, structural element, or structural system to sustain inelastic deformation prior to collapse, without significant loss in resistance, it is a paramount safety characteristic of structures, especially those built in seismic zones. Ductility plays a major role in statically indeterminate structures by allowing redistribution of over-stresses from one section to another, thus delaying failure. So far many researchers have assumed that, since with FRP tendons large deflections can be achieved, ductility requirements can be satisfied. This is a serious and dangerous assumption. Large deflections may be due to the low modulus of the reinforcing system, and may correspond to large amounts of elastic energy stored in the system. The release of such energy at failure could be devastating to the structure and its users, and should raise serious concern (Naaman, 1993).

The linear elastic and brittle behaviour of unidirectional fiber composites is the main reason for a nonductile behaviour of concrete structures reinforced or prestressed with FRP reinforcement. The elongation at rupture is insufficient for the theory of plasticity for design of concrete structures to be applied to structures reinforced with bonded FRP and to give pre warning before failure of the structure. FRP materials do not exhibit yielding before failure. Therefore, as pseudo yield stress for the FRP reinforcement has been suggested by some researchers by applying a reduction factor, ϕ, to the ultimate capacity of FRP. Dolan (1990) suggested a single lower ϕ factor of 0.7 to the ultimate tensile capacity of FRP rod to account for either the rod failure or the concrete compression failure ($f_y = 0.7 f_u$). Faza and GangaRoa (1991) suggested applying $\phi = 0.80$ to obtain a pseudo yield stress for the FRP reinforcement and $\phi = 0.90$ to obtain ultimate strength from the nominal moment capacity.

The designer may also reduce the prestress to provide more ductility, or increase the tendon capacity to provide additional strength. At present, a 60% of the FRP reinforcement tensile strength is considered adequate for the initial prestressing force introduced in the tendons.

2.6.3 Anchorages for Prestressing

Fibers are strong in their direction, but sensitive against lateral pressure, surface injury, etc. Transverse strength is comparably low. Such effects, however, occur at the prestressing tendon's anchorage or during handling on site. They may cause premature failure of FRP tendons.

Although many anchoring devices have been developed with ordinary prestressing systems, these devices cannot be applied directly to FRP tendons, since the surface of FRP tendons is very delicate compared to that of prestressing steel. When used for prestressed concrete, particularly post-tensioned concrete, the main challenge is to obtain an anchorage system that will ensure full development of the FRP reinforcement strength. The low shear strength of composites limits anchorage options to hold tendons.

The design of an anchorage must take into account the brittleness of fibers and their sensitivity against transverse pressure. Anchor equipment varies with FRP reinforcement, and performance is confirmed by experiments. Performance must ensure 95% or more of the tensile strength of FRP reinforcement.

FRP are sensitive to transverse pressure when subjected to high axial stress. High transverse pressure and surface notching reduce the effective tensile strength of FRP. The very high ratio of axial to lateral strength of the FRPs (as high as 30:1) translates into a need to rethink and redesign the anchoring system for cables made of such materials. The conventional systems used for steel tendons cannot ensure a reliable anchorage system, and result in premature anchorage zone failure, and cannot be adopted. The principal goal is to achieve a distribution of the anchoring stresses such that failure of the cable will take place outside the anchoring zone. The tendon anchorages have to be designed in such a way that the tensile strength of the FRP is not significantly reduced by anchorage effects when subjected to static and dynamic actions.

Some tendons are supplied by the manufacture with their anchorage at the ends as preassembled units. Anchorages cannot be replaced in the field without replacing the entire tendon/anchorage unit. The attachment of anchorages to FRP prestressing tendons is neither as straightforward nor as well-developed as it is in the case of steel tendons. Because of this, the anchorages are sometimes attached to a group of prestressing bars in the factory under controlled conditions. The bars are already encased in the ducts before the anchorages are fixed at both ends of the carefully-measured length of the bars. This technique was used in the Marienfelde bridge in Berlin.

Essentially some types of post-tensioning anchorages have been developed which differ with respect to the force-transfer mechanism. Some of the available anchorage systems include: clamp anchorage; wedge-bond anchorage; bond anchorage; split wedge; plug and cone (or barrel and spike); resin sleeve; resin potted; soft metal overlay; end wrap. Also other mechanisms or combinations of several can be applied.

2.7 Applications of FRP Reinforcement to Civil Engineering Structures

As mentioned earlier, FRP is not a new material in engineering. It has been used for decades in mechanical and aerospace engineering, and very often for military purposes. Over the last past years, there has been some increase in the use of these materials for reinforcing concrete, with the possibility of strengthening existing structures. The reason for this development is that FRP can offer the potential for eliminating many problems associated with aggressive environmental influences which result in corrosion of metallic reinforcement. At the same time FRP exhibit several other interesting properties as discussed in the previous sections.

Many structural applications have been suggested for FRP, bridge decks, parking garages, all marine and water exposed structures such as piers, oil platforms, pontoons, quays, artificial islands, railway ties, structures exposed to chemicals or other harsh environments. The majority of applications to date have been in bridges built in Canada, Japan, Germany, and the USA. Bridge types ranging from simply-supported slabs to the most sophisticated systems such suspension and cable-stayed bridges can benefit from FRP's corrosion resistance. Radar stations, antenna systems, telecommunication facilities and metro-rail superstructures can benefit from the electromagnetic resistance of FRP. Also they have been used to reinforce ground anchors and piles, foundation beams and slabs, masonary walls, railways sleepers, rehabilitation projects. Great span suspension bridges or cable-stayed bridges can only be realized applying carbon fiber tendons. Glass fiber bars are an economical alternative as rock reinforcement.

With increasing interest and research efforts, the use of advanced composite materials in civil engineering applications is on the rise. Over fifteen highway and pedestrian bridges have been built in Japan, Europe and Canada using Fiber Reinforced Plastic (FRP) bars. Most of GFRP and AFRP structures were built in Europe, China and the United States, while CFRPs have been studied extensively in Japan and more recently in Germany. Some of the bridges built with FRP reinforcement are reviewed below.

2.7.1 Applications of Fiber-Reinforced-Plastics in Bridges

The first Glass Reinforced Plastic (GRP) highway bridge was constructed in Bulgaria in 1981. The Chinese completed their first GRP highway bridge in 1982 in Beijing known as Miyun Bridge, the total weight is reported to be 1/5th that of a corresponding design of reinforced concrete Tee-beam construction. They claimed that the construction of this bridge proves that a GRP highway bridge is both technically feasible and economically competitive due to the following advantages: reduced weight, which allows for longer spans, simpler construction with a shorter construction period and no major equipment required, suitable to regions of frequent high intensity earthquakes, and a non-corrosive nature which is necessary in corrosive environments. The second pedestrian bridge was completed in 1988 in downtown Chongqing, China, composed of eight GFRP deck girders with 4.3m wide and 0.9 m deep, four 19 m long and four 9 m long, and suspended from reinforced concrete rigid frames by means of high strength steel wire. The GFRP portion of the project amounted to 42% of its total cost, and cost less than a steel bridge of the same type due to lack of corrosion. The bridge is free for 40 years of maintenance.

The Ulenbergstrasse Bridge in Düsseldorf, first European roadway bridge to use GFRP prestressing strands (Polystal tendons) was completed in 1986. It was the world's first large-scale use of prestressing tendons made of glass fiber reinforced composite material. The fiber composite consisted of glass fibers type E, a polyester resin matrix coated on-line with polyamide for protection against mechanical and chemical attack. Another main feature of the Ulendergstrasse Bridge is the application of the so-called 'Intelligent prestressing tendons" by integration of optical fiber sensors and copper wire sensors into a selected number of composite bars. The bridge has two continuous spans of 21.3 m and 25.6 m (69.9 and 84.0 ft.), and is 15 m wide. The GFRP consisted of 59 Polystal internal tendons, each composed of nineteen 7.5 mm (0.3 in) diameter tendon-rods, which provide tensile working forces of 600 kN per post-tensioned unit. In this case of internal prestressing system, the replacement of the entire tendon is not a practical solution, nor is the replacement of a single bar. It is interesting to note that in this bridge a fairly large number of empty ducts are incorporated to accommodate another prestressing system in case something goes wrong with the ACM one, which was used for the first time in a bridge. These ducts could also be used routinely in the future, to enable remedial measures to be taken in case of bar slippage or breakage. The slab was 1.75 m deep cast in place with steel reinforcement (Wolff and Miesseler, 1989, 1991, 1992, 1993).

The Marienfelde Bridge in Berlin completed in 1988 is the first structure to be built in Germany since 1945 with external prestressing without bond. It is a two span 27.61 m and 22.98 m (90.5 and 75.1 ft.), continuous double Tee section beam providing a pedestrian walkway and a bridle path. Seven external Polystal tendons each composed of 19 glass fiber bars are located between the beam webs, each tendon has a working load of 600 kN, 7.5 mm (0.3 in.) diameter, and is covered with a 0.5 mm (0.02 in.) thick sheathing. The prestressing tendons are composed of individual fiber-glass bars. Each bar comprises 60000 single fibers which are oriented strictly in one direction. The tendons run externally between the two main bridge beams, around each of the two transverse beams in the bay sections and then upwards along the central column over the transverse beam. It was reported that four bars out of a group of 19 in a tendon slipped out of the anchorage during prestressing. The treatment for bar slippage was not very difficult, because the external prestressing system readily permitted replacement of the entire tendon, complete with the duct and anchors, at both ends.

The Ulenbergstrasse Bridge and the Marienfelde Bridge, had chambers behind the anchorages at each end of the span, they were made especially large to facilitate monitoring of the instruments installed in the bridge superstructures.

The Schiessbergstrass Bridge in Leverkusen, is composed of three spans 16.3 m, 20.4 m, and 16.3 m, with a total of 27 Polystal prestressing tendons.

The A19 Tees-Viaduct in northeast England, is a composite steel-concrete bridge with a span of 117 m the world's first major bridge enclosure completed in 1987, it is a GFRP cellular construction system in which the rate of corrosion of the plate girders was reduced to 94%. To date, the enclosure represents the largest use of advanced composite materials for a bridge anywhere in the world.

The BASF bridge in Ludwigshafen in Germany is a prestressed roadbridge. The bridge girder has space for two lanes, and the total length is 85m. Four CFRP cables were used in conjunction with 16 conventional steel cables as internal unbonded post-tensioned cables. Each cable consisted of 19 CFRP strands of 12.5 mm diameter, with a prestressing force of 70 kN were applied on each strand. The die-cast/wedge system was chosen for applying prestress by post-tensioning the bridge girder. All cables are guided through tubes embedded in the concrete, steel tendon tubes are filled with concrete after tensioning, while the CFRP tendon tubes were not filled to allow inspection, exchange of cables and data collection.

Chapter 3

Strengthening of Concrete Members using External Carbon Fiber Reinforced Plastic Cables

3.1 Introduction

As mentioned earlier in Chapter 1, cold weather necessitates the use of de-icing salts on roads and bridges. These salts cause corrosion of the steel reinforcement and hence rapid deterioration of the reinforced concrete structures. Corrosion of steel is the most dangerous source of deterioration of concrete structures. It reduces their service life and results in costly repairs and safety hazards. The cost of correcting corrosion-induced distress in bridges is high compared to the capital cost of the structures. The rapid deterioration of old structures requires invention of new methods and materials for rehabilitation and strengthening.

One of the most efficient techniques for strengthening old structures is by applying additional prestressing using external cables located outside the section of structural members. External prestressing can be used to restore the strength and durability to damaged concrete girders and to increase their load-carrying capacity. It has been increasingly considered in the construction of new structures, particularly bridges and has become a primary method of rehabilitation and strengthening of old structures.

External prestressing has the advantage of reducing dead load of structures due to the reduction in web thickness, saving the construction work, easy replacement of old or damaged cables and reducing prestressing losses. This type of prestressing can be applied to various types of structures including reinforced and prestressed concrete, masonry, steel or composite structures. In externally prestressed concrete members, the stress in the cables increases linearly with the increase in the deflection at the service load level. At ultimate it is, important to ensure sufficient level of prestressing which allows for excellent capability of plastic deformation and ductile behaviour. Therefore, it is important to investigate the deformation behaviour of externally prestressed members throughout all stages of loading. Externally prestressed steel tendons, however, should be protected to avoid corrosion. Insufficient protection and lack of maintenance may reduce the service life of the structures.

The only effective method for avoiding corrosion is to replace the steel with a non-corrosive material. Use of Fiber-Reinforced-Plastics (FRP) reinforcement appears to be a feasible solution for this problem. Their non-corrosive characteristics make them attractive to be used as external prestressing for repair, strengthening and rehabilitation of deteriorated structures. Their high strength-to-weight ratio can provide high prestressing forces, good fatigue properties, and low relaxation losses which can increase the service life and the load carrying capacity of concrete structures. Their use has been rapidly increasing in building new structures and in strengthening and rehabilitation of old structures.

This chapter is concerned with the experimental investigation conducted at Concordia University Structures Laboratory into the behaviour of reinforced and partially prestressed concrete members strengthened with externally prestressed carbon fiber reinforced plastic cables. In the following section, an extensive review of previous research work carried out on concrete members reinforced or prestressed with fiber reinforced plastic materials is presented. The objectives, details and testing procedure of the experimental program carried out in the present investigation are described in the remaining sections of the chapter. The results of this experimental program are presented and discussed in the following chapter.

3.2 Literature Review

Fiber reinforced plastic (FRP) materials have been used in concrete structures as reinforcing bars or prestressing cables. Previous research conducted on concrete structures prestressed with FRP cables can be divided into the following three categories:

3.2.1 Studies on Prestressing with Internal Bonded FRP Tendons

Naaman, Tan, Jeong and Alkhairi (1993) studied the flexural behaviour and strength of pretensioned partially prestressed concrete beams using Tokyo Rope's Carbon Fiber Composite Cables. Two 10 foot T-beams were designed with global reinforcing index of 0.11 and partial prestressing ratio (*PPR*) of 0.28, and tested under third-point loading. In addition to the CFCC strands, the beams contained conventional reinforcing bars. The first beam was prestressed with 2 CFCC 1x7 strands 12.5 mm in diameter, while the second beam had 4 CFCC strands placed in two rows, the lower 2 strands were not prestressed. The beam curvature, load-deflection response, cracking pattern and crack width, strain in non-prestressed reinforcement at the critical section with increasing the load, ultimate strength and slip of the strands, and failure modes were investigated and compared to results obtained from similar tests using prestressing steel strands. The test results showed that the load-deflection response of the beams prestressed with CFCC strands generally had a trilinear ascending branch with decreasing slope up to the maximum load. The initial portion extended from zero load to first cracking (uncracked state of behaviour); the second linear portion extended from the first cracking load to yielding of the reinforcing bars (the cracked linear elastic response); the third linear portion extended up to the maximum load at which failure of the FRP tendons occurs. From the tests, deflections and crack widths were generally small but increased rapidly upon yielding of the non-prestressed steel reinforcement.

Naaman et al. observed that the post-peak load-deflection response of the beams was characterized by rapid step-wise decrease in the load due to successive failures of the CFCC strands, and stabilization at about the load-carrying capacity of the remaining steel reinforcing bars. The presence of the reinforcing bars also helped the beams sustain large deflections before crushing of the concrete in the compression zone, and thus, residual strength and ductility was provided by the non-prestressed reinforcing bars. Longitudinal cracks in the constant moment region at the level of the CFCC strands in the first beam, and at the level of nonprestressed CFCC stands in the second beam, occurred due to rupture of he CFCC strands which released sufficient fracture energy to and led to the longitudinal cracks in the concrete. In a comparison with the behaviour of beams prestressed with conventional steel strands, Naaman et al. found that for the same global reinforcing index, the cracking load was lower, the crack width was larger and the beam stiffness after cracking was smaller for the beam prestressed with CFCC strands than for those prestressed with conventional steel.For the same partial prestressing ratio, the behaviour of the beams prestressed with the two types of strands can be very similar.

Guha-Thakurta et al. (1994) and Abdelrahman, Tadros and Rizkalla (1995) conducted an experimental program to examine the various limit states behaviors, ultimate capacities, and failure modes of 1:3.3 scaled model of the first smart highway bridge, prestressed by CFCC and Leadline rods. The bridge is a 22.83 and 19.23 m two span continuous skew bridge, located at Center Street/Beddington Trail in Calgary, Alberta, Canada, and consists of a total of 26 bulb-Tee section precast prestressed concrete girders, 13 for each span. Four pretensioned concrete T-beams with a total length of 6.3 m (clear span of 5.8 m), overall depth of 330 mm and flange width of 450 mm, and with the same span-to-depth ratio as the bridge girders were tested. Two beams, one with CFCC and the other with Leadline rods were tested monotonically to failure and the other two beams were subjected to cyclic loading to examine the fatigue failure with maximum load equal to the cracking load of each beam, and a minimum load equal to 70 percent of the cracking load. Both types of tendon were jacked to 60 percent of the guaranteed strength provided by the manufacturer. The test results of the beams prestressed by both Leadline and CFCC, tested under static loading conditions, behaved linearly up to cracking and after cracking with reduced stiffness up to failure because of the linear characteristics of the CRRP. When the beams were unloaded at a load equal to 95% of the predicted failure load, it was observed that almost all cracks closed and the permanent deformation was almost zero, and the original stiffness for the beams was restored during reloading up to failure. Prior to failure, flexural cracks extended to the top flange of the beams, and a major horizontal crack was observed at the onset of failure. The failure was caused by rupture of the cables. The failure load was about 30 and 60% higher than the predicted for the beams prestressed with CFCC and Leadline, respectively based on the guaranteed strength of CFRP tendons.

The results of the cyclic loading tests showed that the beam prestressed with CFCC sustained two million cycles with no sign of reduction in stiffness at a different number of cycles because of the elastic behaviour of the CFCC, and that the load deflection response was perfectly elastic during and after completion of the two million cycles. Before application of the cyclic load on beam prestressed with Leadline, the beam was initially loaded close to the failure load measured from the static test. As a result, flexural cracks formed and propagated upward to the upper flange. From the test, it was observed that the stiffness of the beam was reduced by increasing the number of cycles, due to the initial extensive cracking of the concrete. After completion of the two million cycles, the beams were loaded statically up to failure, there was no reduction in the load carrying capacity of the beams and their behaviour was identical to the first two beams. The failure load for beams with CFCC and Leadline, respectively was 95 and 100 percent of the failure load of the beams tested under static load.

Abdelrahman and Rizkalla (1995a and b) studied the serviceability and investigated the modes of failure of eight concrete beams prestressed with Leadline rods and two beams prestressed with conventional steel strands. All test specimens were 6.2 m long (5.8 m clear span) simply supported span and 330 mm in depth representing typical span-to-depth ratio used for bridge girders. The tested beams had T-shape cross-section and two different flange widths (200 and 600 mm) were used. The beams were tested using two static concentrated loads cycled three times between a lower level equivalent to 80% of the cracking load of the beams and an upper level equivalent to 60% of the predicted strength of the beams. The latter load level was equivalent to 1.5 to twice the cracking load depending on the prestressing level. The authors reported that the aim of the cyclic loading at service load limit was to study the deflection, after reduction of the beam stiffness and the cracking behaviour after stabilization of cracks. The parameters considered in their investigation included the prestressing ratio, the degree of prestressing and the distribution of the CFRP bars in the tension zone to study their effects on the cracking behaviour. The various limit states flexural behaviour considered in their study included the deflection before and after cracking, crack pattern, spacing and width, strain distribution and failure modes. The jacking stresses used for the rods ranged from 50 to 70% of the ultimate guaranteed strength of the Leadline as specified by the manufacturer. They found that beams prestressed by Leadline jacked to 70% of the guaranteed strength had 4 to 8% higher ultimate load than the beams prestressed by Leadline jacked to 50% of the guaranteed strength. Also, the deflection at failure was about 11 to 18% less than beams jacked to 50% of the guaranteed strength.

The test results indicated that while the bond strength of the Leadline at the prestress transfer zone was comparable to that of steel strands, the flexural bond strength of the Leadline was less than that of steel strands. Abdelrahman et al. attributed this behaviour to the fact that the Leadline has lower modulus of elasticity which for the same load level leads to larger longitudinal deformations and hence larger transverse deformation than steel knowing that Poisson's ratios of Leadline and steel are almost equal. They reported that the higher transverse deformations improve the bond strength at the transfer zone due to the lateral expansion of the rod which creates wedge action known as the Hoyer effect.

The test results also indicated stabilization of the flexural cracks of the beams prestressed by the Leadline rods at significant lower strain level than beams prestressed by steel strands. The number of cracks in the beams prestressed by Leadline was less than that in beams prestressed by steel strands, and the spacing between cracks in the beams prestressed by Leadline was larger than the beams prestressed by steel strands. This was attributed to the lower flexural bond strength of the Leadline rods. Beams prestressed by Leadline rods had similar stiffness to beams prestressed by steel strands before cracking, while after cracking, the stiffness of the beams prestressed by Leadline was less than the beams prestressed by steel. The beams prestressed by Leadline rods behaved linearly before and after cracking but with reduced stiffness after cracking up to failure. This was attributed to the linear elastic characteristics of the Leadline rods. It was also noticed that because of the lower modulus of elasticity of the Leadline, the depth of the neutral axis, and hence the compression zone, decreased more rapidly with the increase in applied loads than in the beams prestressed with steel.

In the beams with 600 mm flange width prestressed with Leadline, failure was caused by rupture of the prestressing reinforcement, with ultimate load 37% higher, and ultimate deflection 50% lower than the beams prestressed with steel. The beams with 200 mm flange width prestressed with Leadline, failure was due to crushing of the concrete; the ultimate load was 27% higher and the ultimate deflection was the same as the beams prestressed with steel strands.

In addition to the experimental program, the authors used an analytical model based on the strain compatibility and equilibrium approach to predict the behaviour of the beams. They also proposed a model to evaluate the ductility of the beams prestressed with CFRP bars. They used the following four methods to predict the deflection taking into account the tension stiffening: 1) the effective moment of inertia modified by Tadros, Ghali and Meyer (1985); 2) the CEB-FIP Code (1993); 3) the mean curvature calculated based on strain compatibility including the tensile strength of concrete, and 4) a simplified method using the effective moment of inertia approach. For the first three methods, the deflection was calculated based on integration of the mean curvature along the span of the beam. Their analysis showed that using any of these methods, the calculated deflection was in excellent agreement with the measured deflection, while the simplified method could be used as a good tool for preliminary design.

Fam et al. (1995) conducted an experimental program to examine the behavior of 1:3.6 scaled model of highway concrete bridge girders, reinforced for shear using two types of CFRP reinforcement. The bridge is located over the Assiniboine River, Parish of Headingley, Winnipeg, Manitoba, Canada, and consists of five spans, 32.5 m each covering a total length of 165.1 m. All the bridge girders are precast pretensioned and simply supported, with an I-section AASHTO type transversely spaced at 1.8 m and supporting 187 mm thickness deck slabs. The two types of CFRP were CFCC and Leadline rods. Six 9.3 m long I-shape cross section pretensioned beams of an overall depth of 500 mm were tested with the same prestressing level and span to depth ratio of 17.8 of the bridge girders. The main aspects of this testing program were the use of CFRP for shear reinforcements, draping of the prestressing strands, and extending the CFRP stirrups into the slab to provide the dowel action similar to the deck slab of the bridge (to simulate the composite behaviour). Different stirrup sizes were used to study their effect on the shear resistance in terms of diagonal crack width, crack distribution and the dowel action. Also the performance of prestressed concrete I-girders totally reinforced with FRP reinforcement for shear and prestressing was evaluated in terms of flexural cracks, deflection, and ultimate resistance. The efficiency of the prestressing CFRP tendons draped with an angle of 4° was also examined. The test results showed that draping of CFRP tendons was feasible, however, it was found that the residual stresses at the draping points could trigger the flexural failure. The anchorage system used consisted of steel coupler used to couple the CFCC and Leadline prestressing reinforcement to conventional steel strands to minimize the cost of CFRP and facilitate using the same jacking system of the existing prestressing equipment.

From the test results, all beams behaved linearly up to the initiation of the first set of cracks and after cracking due to the linear characteristics of the CFRP with 70% reduction in the stiffness of the beams prestressed by CFCC and 65% reduction of the stiffness of the beams prestressed by Leadline up to failure. No slip was observed in the prestressing strands neither at the end of the beam nor between the girder and the top slab up to failure for all tested beams. The authors reported that the strength of CFRP stirrups was adequate to transfer the stresses between the top slab and the beam by dowel action. The beams prestressed by Leadline failed at higher load level in comparison to the beams prestressed by CFCC due to the higher tensile strength of the Leadline. Also, failure of the beam reinforced by Leadline stirrups was by shear rather than flexure because the Leadline rods have higher ultimate tensile strength than the CFCC. Shear resistance of Leadline stirrups was limited to a maximum strain of 0.85% which represent 45% of the uniaxial tensile strain of the reinforcement. The authors also found that shear failure was very brittle and caused breaking of the beam to several pieces due to the energy released from rupture of the longitudinal prestressing tendons.

Arockiasamy, Zhuang, and Sandepudi (1995) studied experimentally the durability and flexural behaviour of nine concrete beams prestressed with CFRP tendons and exposed to seawater/alkaline environments. Three beams were tested as control specimens and six beams were pre-cracked by loading up to 60% to 70% of the predicted ultimate failure value before immersion in seawater/alkaline solution in order to accelerate the degradation. Three of those six were subjected to wet/dry cycles in seawater. The typical cycle period was two to three days. The other three beams were subjected to an alkaline solution with pH value of approximately 14. The exposed beam specimens were tested after three months, six months and nine months of immersion. The tests results showed that the seawater/alkaline environments did not affect the flexural strength of the pre-cracked beams when exposed to seawater/alkaline solution. The test results compared well with the predicted values based on the conventional theory for concrete beams prestressed with steel tendons.

3.2.2 Studies on Prestressing with Internal Unbonded FRP Tendons

Very few investigations into the behaviour of concrete members prestressed with internal unbonded FRP tendons have been found in the literature. In an investigation into the effects of bond between the prestressing tendons and the concrete on the behaviour of concrete members prestressed with CFCC tendons, Mutsuyoshi, Machida, and Shiratori (1990) tested seven concrete beams prestressed with CFCC with different degrees of bond between the cables and the concrete under different levels of prestressing. Three degrees of bond strength were used: perfect bond, reduced bond and zero bond (i.e. unbonded cables). The reduced bond strength was achieved by wrapping the cables with vinyl tape. The level of the prestressing force introduced to the cables varied between 40 to 60% of the tensile strength of the cables. The behaviour of the CFCC prestressed beams was compared to that of geometrically identical beams, but prestressed with bonded steel tendons, tested under the same conditions.

From their tests, Mutsuyoshi et al. (1990) observed that before initial cracking, the stiffness and behaviour of the beam prestressed with bonded CFCC under high prestressing force were similar to those of the beam prestressed with steel. However, the CFCC-prestressed beam had higher ultimate strength and failed by crushing of the concrete in the compression zone whereas the beam prestressed with steel reached its ultimate state with failure of concrete after yielding of the steel tendon. The CFCC- prestressed beam with reduced bond strength of the cables exhibited large deformations and showed a more ductile behaviour up to failure (which took place without breakage of the CFCC) in compression with the similar beam prestressed with fully bonded CFCC. In the latter beam, failure took place by breakage of the CFCC accompanied by a sudden decrease in the load. The beam with completely unbonded CFCC exhibited a more ductile load-deflection response up to failure and its maximum strength was lower than that of the beam with fully bonded CFCC. The authors concluded that CFCC can be a suitable replacement for conventional steel tendons provided that the magnitude of the prestressing of the CFCC by wrapping with a tape is an appropriate method to avoid sudden failure by breakage of the CFCC.

Kato and Hayashida (1993a and b) reported on the effects of improving the flexural characteristics and ductility in unbonded prestressed concrete beams when using CFRP tendons, and the bending fatigue characteristics of bonded prestressed concrete beams with CFRP tendons. They conducted tests on three specimens for static bending and three for fatigue bending, with a various types of CFRP tendons (strand type, bar type) and profiles of tendon (straight, curved). Kato et al. compared the behaviour of bonded type prestressed concrete beams using post-tensioned CFRP with that of conventional prestressed concrete beams and found that the flexural cracking loads were roughly the same since they were mainly affected by magnitude of the prestressing force. However, the yielding load was slightly larger for the conventional prestressed concrete beams due to the higher value of Young's modulus of the tendons. On the other hand, when CFRP tendons were used, the flexural strength increased even after flexural yielding, and the failure was determined not by crushing of concrete in the compression zone, but in almost all cases by rupture of the tendons and the tension reinforcement. Deformation of beams at ultimate were found to be smaller than in the case of conventional steel tendons. The main reasons for this are the absence of yield points for FRP tendons and the considerable smaller elongation at rupture as compared to that of conventional steel.

The flexural fatigue tests of bonded beams showed that for specimens subjected to repetitive loading in excess of one million cycles the flexural failure and load-displacement curve were similar to those obtained under static bending. No reduction in the ultimate flexural loads due to repetitive loading were observed. All specimens failed by rupture of the CFRP tendons. The number of cracks and their widths were roughly the same for each specimen under the two types of loading. The authors also found that the cracking and the ultimate flexural load can be roughly estimated using the prestressed concrete standard equations. Similar to Mitsuyoshi et al. (1990), Kato and Hayashida (1993a and b) came to the conclusion that the ductility at ultimate of concrete beams prestressed with CFRP tendons can be improved by the use of unbonded tendons. They also concluded that the effects of fatigue loading were not recognizable which indicated that concrete beams prestressed with CFRP tendons have good fatigue characteristics.

Maissen and De Smet (1995) compared experimentally the behaviour of beams prestressed with CFRP and beams prestressed with steel under static loads. The load-deformation characteristics, the behaviour at ultimate and ductility were of major interest as they can provide sufficient warning prior to failure. The CFRP tendons used were 12.5mm Tokyo Rope CFCC strands. Three 6 m span T-section beams were tested: one with bonded CFRP tendons, one with unbonded tendons, and one with bonded steel tendons.

The specimens were simply supported and loaded with pairs of relatively close point loads. The prestressing level in the CFRP prestressed concrete beams was 50%, while in the steel prestressed concrete beam was 70%. A comparison of the behaviour of bonded and unbonded CFRP prestressed concrete beams showed that the unbonded beam has a considerable lower capacity than the bonded beam. The deformation capacity however is only insignificantly smaller. From the test results the following conclusions were drawn:

- The load-deflection response showed that both specimens with bonded CFCC and steel tendons exhibited a ductile behaviour before failure. Because of the low level of prestressing (50%) in the CFRP strands and the strain reserves large deformations occurred prior to failure, even though the CFRP strands themselves exhibited perfectly elastic behaviour.
- The beams prestressed with steel showed much larger ductility than the CFRP beams because of the plastic behaviour of the steel strands.
- The deformations of the beam prestressed with CFRP prior to failure were sufficiently large to give ample warning prior to collapse. Therefore the failure could still be classified as ductile.
- The beam with unbonded tendons had fewer cracks and lower capacity than the beam with bonded tendons. This was due to the concentration of rotations at the few crack locations, which caused local increases in concrete compression and led to a premature failure of the beam by crushing of concrete in the flange. Thus the capacity of the specimen was not controlled by the ultimate strength of the prestressing strands. On the contrary, failure occurred before the full deformation of the strands could be utilized. Therefore, this does not apply only to CFRP prestressed concrete, but to unbonded prestressed concrete beams in general.

3.2.3 Studies on Prestressing with External FRP Tendons

In a comparative study between the behaviour of concrete members prestressed with internal unbonded and external prestressed FRP tendons, Burgoyne (1992) tested two beams prestressed with parallel-lay aramid (Parafil) ropes. The first beam was stressed with a single straight internal unbonded tendon passing through a plastic duct on the centerline of an I-shaped cross section. The tendon was stressed initially to 70% of the nominal breaking load. The second beam was post-tensioned by two external tendons, one on each side of a T-shaped cross section. The tendons were draped at two points close to the loading points making a 4.75° angle with the longitudinal axis of the beam.

The tests showed that both beams exhibited considerable curvature at a virtually constant ultimate load, with large cracks forming at the bottom of the beam. Failure of both beams occurred by crushing of concrete in the top flange. Slight differences in the final mode of the two beams were observed. The beam with internal unbonded tendon did not collapse completely. Crushing of the concrete in the top flange was followed by compressive failure passing down through the web and into the top of the bottom flange. This was accompanied by a reduction in the load as the beam was trying to carry its own weight on a steadily reducing section. The bottom flange did not fail, however, and remained axially prestressed. The externally prestressed beam, on the other hand, failed suddenly and completely with a total loss of prestressing. This was attributed to the fact that, in the second beam, the tendon was outside the bottom flange, which could thus deflect leaving the tendon in its original position relative to the ends of the beam, while in the first beam the tendon was constrained in the bottom flange. Based on this observation, Burgoyne proposed that if the tendons were external to the concrete, they should pass through loose rings so that, in the event of failure, the tendons would be forced to deflect with the beam. This may ensure that failure occurs in a more controlled manner as in the first beam, rather than in a sudden manner as in the second beam. In addition, the compression zone of the concrete should be provided with confining reinforcement to increase the ductility of the concrete in that area.

Burgoyne (1993) argued that since FRPs are brittle materials which do not show plastic yielding, it is not desirable to have FRP tendons bonded to the concrete. They should be used only as internal bonded or external prestressing tendon. He explained that when flexural failure starts in a beam, cracks open and become wider under increasing load. When a tendon is bonded to the concrete, it will be forced locally to have very high strains. If the tendon is ductile, such as steel, it will yield without failure. However, if the tendon is brittle, as in the case of FRP, it will snap under high strain and lose all its tensile force.

Mutsuyoshi and Machida (1993) conducted an experimental investigation into the behaviour of T-shaped concrete beams prestressed externally with three different types of tendons. Three beams were prestressed with carbon fiber composite cables (CFCC), two were prestressed with aramid fiber reinforced plastic (AFRP) cables, and one was prestressed with conventional steel cables. The external cables were draped at two saddles points and prestressed with a force of less than 50% of the nominal tensile strength of the cable. Two bending angles of the cables, 7.1° and 11.3° at the saddles were used in order to investigate their effects on the behaviour of the beam and the cable. The load-displacement curves obtained under static and fatigue loading on the beams prestressed with CFRP and AFRP indicated similar behaviour to the beam prestressed with steel. However, two different modes of failure were observed. The beams prestressed with steel and AFRP failed in compression of the concrete, while the beams prestressed with CFRP failed by simultaneous crushing of the concrete and breakage of the cables. Mutsuyoshi and

Machida (1993) also found that the breaking strength of the CFRP cable was greatly influenced by the bending angle at the saddle points or deviators. A reduction of about 20% in the average breaking load from the uniaxial tensile test was obtained. They recommended such reduction in strength be taken into account in the design of concrete members prestressed with CFRP tendons.

From the results obtained from their test, Mutsuyoshi and Machida (1993) concluded that the ultimate flexural strength of concrete members prestressed with FRP external cables cannot be obtained accurately from the equations available for unbonded prestressed concrete members using steel cables. They proposed an analytical method to evaluate the flexural strength and the force in the cables in externally prestressed members with FRP. Their method was based on compatibility condition on deformations, that is, the elongation of the cable equals that of the concrete located at the same position at the ultimate state. However, they indicated that their method is too complicated to be used in design and that a simpler method needs to be established. Another experimental investigation similar to the one of Mutsuyoshi et al. (1993) was carried out by Tsuchida et al. (1993) and resulted in similar findings.

Saeki et al. (1993) conducted an experimental study into the rehabilitation and strengthening of damaged concrete beams by external post-tensioning of parallel-filament aramid rope (known commercially as Kevlar). They tested twelve 3 m long reinforced and prestressed concrete beams with rectangular cross-sections. The tests were divided into two series of static and fatigue tests under three-point loading. Flexural cracks were produced into the beams to simulate damaged before strengthening by the external prestressed aramid ropes. Some of the cracked beams were repaired by injection of epoxy resin through the cracks before the beams were externally prestressed. The static loading tests were conducted to measure the ultimate strength, deflection, the tensile force in the aramid cables, the strain in the reinforcing bars and the concrete, and the width of cracks. The fatigue tests were conducted only on the prestressed concrete beams to evaluate their ultimate strength and rigidity as well as the behaviour of the external cables.

The deformation and ultimate strength obtained from the static loading tests confirmed that the beams were reasonably upgraded and strengthened. The tests also showed that epoxy injection was an effective means of cracks repair and played a vital role in restoring the rigidity of the damaged beams to its initial level. The fatigue tests indicated no significant difference between the rigidity of the beams before and after testing. After 2 million cycles of loading, no beams were found fractured and the ultimate flexural strength of the beams after testing was almost the same as that before testing. The authors also claimed that the repair of cracks was effective in decreasing the loss of prestressing.

Miyamoto et al. (1994a) developed a design concept and investigated the application of external prestressing with FRP tendons for the construction of new prestressed composite bridges and for strengthening and repair of old bridges. Their investigation focused on the effects of external FRP tendons on both the elastic and plastic behaviour of prestressed composite girders. The investigation included the effects of several parameters on improving the performance of this type of bridge. The parameters considered included the cable arrangement and eccentricity, the material properties of cable including the elastic modulus, the tensile and yield strength, the elongation, the cross-sectional properties, the prestressing force, and the prestressing procedure. The authors developed an analytical model based on the incremental deformation method applied on a standard composite girder strengthened by application of external prestressing to the steel girder. Their proposed design concept was based on the degree of improvement in load-carrying capacity and the control of deflection. They concluded from their parametric analysis that the condition of each of the above-mentioned parameters had a different effect on the degree of improvement in performance and, therefore, the cable arrangement and type of cable material have to be properly selected to suit each strengthening purpose. Based on the results of their parametric study, the authors presented a detailed procedure for strengthening purposes. They showed that their strengthening procedure could achieve an increase in the yield load and the ultimate load by 43% and 88%, respectively.

Miyamoto et al. (1994b) verified the above-mentioned strengthening procedure by comparison with the results of tests conducted on the full-scale prestressed composite plate girder with different levels of prestressing force, eccentricity of draped tendons and material properties of tendons (conventional steel and Aramid FRP). Models with draped and straight tendons were analyzed. The external draped tendon was placed below the steel beam in the so called a "queen post" form. The external tendon was anchored at both ends of the beams. In the tests, the draped tendon was in direct contact with the saddle and friction existed between the tendon and saddle. The tests indicated that there were two ways to utilize the effects of prestressing. One was to set an external tendon that would allow application of the greatest possible compressive stresses to the bottom flange of the steel beam in the direction opposite to the stresses created by the external load. The other was to increase the eccentricity of the external tendon to control the deformations or improve the load-carrying capacity of the beam. The effects of the tendon's material properties and eccentricity indicated that a steel tendon tended to provide a yield load and ultimate load greater than those provided by an Aramid FRP tendon. This was because the modulus of elasticity of steel is about three times as large as that of an Aramid FRP tendon. The yield load and the ultimate load were increased by 54% and 55%, respectively for the beam strengthened with steel tendons, and by 50% and 48%, respectively for the beam strengthened with Aramid ropes. The eccentricity and material properties of the external tendon were the major factors that had a significant effect on the improvement in the performance of the composite girders. In the case of both steel and Aramid tendons, a smaller eccentricity led to a larger yield load because the smaller eccentricity produced a larger compressive stress on the bottom flange of the steel beam when a prestress determined on the basis of allowable tensile stress of the concrete slab was applied. The ultimate load was larger for the large eccentricity models because the larger eccentricity provided more resistance of the external tendon against the beam deformation. The authors noted however that the eccentricity had an opposite effect on the yield and ultimate loads. This is particularly important and requires special attention when prestressing is applied to strengthen an existing bridge. Their experimental and analytical studies indicated that a prestressed composite beam has excellent yield load and ultimate load-carrying capacities and that prestressing newly introduced external tendons is an effective technique for strengthening existing composite girders.

Horiguchi, Saeki, and Hata (1995) carried out an experimental into strengthening damaged concrete beams by external prestressing of parallel-filament aramid fiber cables (Parafil type F). Static as well as fatigue tests on T-cross sectional members under 3-point and 4-point bending at nominal and low temperatures were conducted in order to evaluate the effects of temperature on the fatigue properties of externally prestressed concrete beams. The statical loading test showed that temperature is an important factor that affects the behaviour of concrete members prestressed with aramid fiber cables. For example, it was found that there was a significant difference in the mode failure under different temperatures. The mode of failure under normal temperature was a combination of compressive failure of concrete and tensile fracture of the steel reinforcement. It was also found that the statical ultimate flexural strength of the beams improved while their fracture toughness decreased at low temperature. In the fatigue tests, only tension failure of steel reinforcement was observed and the fatigue strength of external prestressing cables decreases at low temperature. The authors concluded that external prestressing with aramid fiber cables was effective in strengthening the tested beams.

Saeki, Horiguchi, and Hata (1995) conducted an experimental study on the plastic deformation properties of rectangular and T- cross sectional beams strengthened by externally prestressed aramid fiber cables to increase the ultimate shear and flexural strengths. The external prestressing was applied after repair of cracks. Saeki et al. examined the effect of the effective prestress level of the external cables on the plastic deformation capacity of the beams. They found that the plastic deformation behaviour of the beams could be ductile when the initial prestress was controlled at a reasonable stress level. They also compared the experimental test results of the ultimate strength and deformation with the numerical values obtained based on Pannel's analysis (1969). The comparison was used in modifying the coefficient of plastic hinge length and the effective depth of the external cable at failure. The analysis of ultimate strength and deformation was performed as summarized below.

The ultimate strength was calculated from:

$$M = bd^2 f_c' \left[\left\{ 1 - \frac{\beta}{\alpha}(q_s + q_u) \right\}(q_s + q_u) + l_p{}^* q_u \right] \tag{3.1}$$

where

b and d are the width and effective depth of the cross section, respectively; f_c' is the compressive strength of concrete; α and β are coefficient equal to 0.8 and 0.4, respectively; q_s and q_u are given by:

$$q_s = \rho_s \frac{f_{sy}}{f_c'} \tag{3.2}$$

and

$$q_u = \frac{q_e + \lambda(1 - q_s/\alpha)}{1 + \alpha} = \frac{\rho_{ps}\xi f_{pu}}{f_c'} \tag{3.3}$$

where ρ_s and ρ_{ps} are the reinforcement ratios of the nonprestressed and prestressed reinforcements, respectively; f_{sy} and f_{pu} are the yield strength and ultimate strength of the two types of reinforcement, respectively; q_e is given by:

$$q_e = \rho_{ps} \frac{f_{pe}}{f_c'} \tag{3.4}$$

with f_{pe} being the effective prestress in the external cable; the coefficient λ in Equation 3.3 is given by:

$$\lambda = \frac{\phi\varepsilon_{cu} E_{ps} \rho_{ps} d}{f_c' l_p} \tag{3.5}$$

in which ϕ is a coefficient equal to the plastic hinge length divided by the depth of neutral axis, kd; ε_{cu} is the ultimate concrete strain at failure; E_{ps} is the modulus of elasticity of the prestressing cable and l_p is the length of external rope between anchorages. The value ξf_{pu} in Equation 3.3 is the stress in the cable at failure ($=f_{ps}$).

In Equation 3.1, l_p^* is given by:

$$l_p^* = \Delta e - \frac{\phi S_l \varepsilon_{cu}}{4d} \tag{3.6}$$

where $\Delta e.d$ is the length from primary reinforcement to the external cable and S_l is the length between deviators.

The ultimate deflection D at mid span is written as follows:

$$D = \frac{L\theta_p}{4} \tag{3.7}$$

where $\theta_p = \phi\varepsilon_{cu}$

The coefficient ϕ for plastic hinge length is unknown when calculating the ultimate strength from Equation 3.1; it can be estimated as follows:

$$\phi = \frac{(\xi f_{pu} - f_{pe})l_p}{E_{ps}\varepsilon_{cu}(1-k)d} \tag{3.8}$$

where

$$k = \frac{\rho_{ps}\xi f_{pu} + \rho_s f_{sy}}{\alpha f_c'} \tag{3.9}$$

Saeki et al. (1995) showed that since ϕ is related to the ratio of neutral axis depth at failure, ϕ can also be related to the sum of $(q_s + q_u^*)$, with q_u^* being $\rho_{ps} f_{pu}/f_c'$ and a lower average value of ϕ can thus be calculated as:

$$\phi = 20 + \frac{1}{2(q_s + q_u^*)} \tag{3.10}$$

They also stated that in determining the allowable effective prestressing, the upper value of ϕ given by Equation 3.11 should be adopted:

$$\phi_c = 35 + \frac{0.8}{(q_s + q_u^*)} \tag{3.11}$$

A critical effective prestress can thus be defined as $(\eta_c \cdot f_{pu})$, with η_c given by:

$$\eta_c = 1 + \frac{\phi_c E_{ps}\varepsilon_{cu}(1-k_c)d}{f_{pu}\, l_p} \tag{3.12}$$

where

$$k_c = \frac{\rho_{ps}\xi_c f_{pu} + \rho_s f_{sy}}{\alpha f_c'} \tag{3.13}$$

$$\xi_c = \frac{f_{pe} + \lambda_c(\alpha f_c' - \rho_s f_{sy})/\rho_{ps}}{f_{pu} + \lambda_c f_{pu}} \tag{3.14}$$

$$\lambda_c = \frac{\phi_c \varepsilon_{cu} E_{ps} \rho_{ps} d}{\alpha f_c' l_p} \tag{3.15}$$

From their tests, Saeki et al. found that the coefficient ϕ decreases from 30 to 20 with the increase in $(q_s + q_u^*)$. This coefficient ϕ is larger than the value of ϕ=10 proposed by Pannell (1969) for unbonded type prestressing. They observed that the ultimate strength and deformation of externally prestressed concrete beams were apt to have comparatively large variation due to the sensitive behavior of plastic hinge formation. They recommended that a lower average value of the coefficient ϕ as be used to ensure safe estimation of the ultimate strength and deformation and that the upper limit value of ϕ be used for the design of the allowable initial effective prestress in order to maintain a ductile failure without cable breaking. The factor η_c is available for estimating the permissible effective external prestress. Their test results also indicated that the deflection and ultimate strength at the stage of plastic hinge formation could be analysed with the modified Pannell's analysis in consideration of large zone plastic hinge and small effective depth of external cable.

Grace and Abdel-Sayed (1996 a and b) suggested a new system to replace bridge superstructures deteriorated mainly by steel corrosion caused especially in the presence of roadway de-icing salts. The proposed improved system consisted of precast prestressed concrete Double-T (DT) girders covered with a deck slab. The DT girders were reinforced with glass fiber reinforced plastic (GFRP) bars and prestressed internally and externally with draped carbon fiber reinforced plastic (CFRP) strands. The CFRP strands used were Carbon Fiber Composite Cables (CFCC) provided by Tokyo Ropes. The behavior and response of the new system was investigated during the following different phases of construction:

- The DT girders were precast, reinforced with GFRP bars and internally post-tensioned with CFRP strands in the factory. The prestressing force in the internal strands were designed to support the dead load of the girder and the deck slab added later on-site. The longitudinal internal prestressing produces upward camber and placed the entire cross section of the DT girders in compression, making them safer to transport.
- In phase two, the DT girders were transported to the site and placed side by side on the support and transversely post-tensioned using internal CFRP strands passing through cross beams and intermediate deviators. Because of internal prestressing used in the fabrication, this system eliminates the need for extensive on-site formwork and shoring, and thus reduces the construction time and cost.
- Phase three consisted of a cast-in-place deck slab over the entire bridge reinforced with GFRP bars and connected to the DT-girders through shear connectors extending from the DT girders into the concrete deck slab to ensure the transfer of the horizontal shear forces. Laboratory tests on a bridge model indicated that 100% interaction between the deck slab and the DT-girders was ensured. Due to addition of the deck slab, the authors observed some hairline cracks created by tension strain at the bottom of the webs in the transverse direction at midspan causing the bridge model to deflect downward. These cracks were completely closed after application of the external prestressing in the final phase.

- In the final phase, externally-draped, post-tensioned CFCC strands were placed to restore the upward camber and to place the top of the deck slab in tension and the bottom of the webs in considerable compression at the middle of the span. The authors reported that the external prestressing closed the cracks formed by the addition of the deck slab. The bridge model was also experiencing an upward camber which was desired to counteract the deflections of the simulated traffic load. The authors indicated that the upward deflection can be controlled by adjusting the level of the prestressing and the size of the externally draped CFCC strands. The presence of shear connection transferred the effect of external prestressing to the deck slab and eliminated any slippage between the DT girder and the deck slab.

The test results indicated that the use and proper implementation of composite rebars and prestressing strands along with the DT cross-sectional girder is a very promising approach to produce a ductile, crack-free and long-lasting bridge system. Also the tested DT bridge had confirmed its ability to sustain repeated load of 60% of its ultimate load carrying capacity for seven million cycles without any significant changes in its dynamic and static characteristics. The authors claimed that this new system should offer superior corrosion resistance when compared to conventional bridge construction, provide savings in construction time and cost, and minimize the need for maintenance.

Jerrett, Ahmad, and Scotti (1996) conducted tests on four concrete beams pretensioned with steel strands and strengthened by external CFRP post-tensioned tendons. The beams were tested under four point loading. The external prestressing used for strengthening was provided by two 8mm diameter CFRP Leadline tendon of 104 kN tensile strength, and 150 GPa elastic modulus, one tendon on each side of the beam. Each tendon was harped at two points below the point loads at an angle of 4.8 degrees at each point. The tendons were bent about a steel harping plate that was cut to a radius of 508 mm. The harping plates were attached to a pair of 51 mm steel tubes that were placed on the bottom side of the beams. The CFRP tendons were attached to the beams using steel saddles at each end of the beam. The test results showed that the average strength increase of the strengthened members was 115% for the beams with single strands and 46% for beams with double steel strands. Midspan deflections at the ultimate load for the strengthened beams were approximately 60% of the corresponding control beam deflection. All beam failures were due to crushing of concrete.

Based on the research work carried out to date on the behaviour of concrete structures reinforced or prestressed with FRP reinforcement, The CHBDC Technical Subcommittee No.16 proposed the first design provisions for fiber reinforced structures for the Canadian Highway Bridge Design Code (CHBDC, 1996). According to these provisions the full capacity of FRP tendons is not full utilized in practice. This is because the provisions specify allowable stresses in the tendons well below their failure loads. For example, the provisions specify that the allowable stress in the tendon at ultimate, f_{ps}, for pretensioned concrete components, computed using a method based on strain compatibility, is not to exceed $0.7f_{pu}$ for AFRP and $0.85f_{pu}$ for CFRP. For straight FRP tendons or curved tendons having a radius of curvature greater than 1200 times the tendon diameter, the provisions specify the maximum stresses at jacking and transfer as given in Table 3.1 For curved tendons with a radius of curvature smaller than 1200 times the tendon diameter of the tendon, the maximum stresses at jacking and transfer are those given in Table 3.1 but reduced by a factor equal to $(0.5\ E_{FRP}\ d_s) / R_t$, where E_{FRP} is the modulus of elasticity of the FRP tendon, d_s is the diameter of the strand, and R_t is the radius of curvature of the tendon. In spite of the above, the provisions require that the anchors for FRP tendons have at least the same nominal capacity as the FRP tendons. This is because anchors having smaller capacity are likely to be inefficient in that they may overstress some fibers and understress others, and this may lead to a premature failure resulting from creep rupture.

It can be seen from the above literature review that two important factors that affect the behaviour of prestressed concrete structures, namely, the partial prestressing ratio and the span-to-depth ratio of the concrete members, were not considered by previous researchers. Therefore, as part of the present research, an investigation has been conducted into the effects of these two important parameters on the behaviour of partially prestressed members strengthened with CFRP external cables. The experimental program carried out for the purpose of this investigation is described in the following section. The results of this experimental work are presented and discussed in the next chapter.

Table 3.1: Maximum Permissible Stresses in FRP Tendons at Jacking and Transfer (CHBDC 1996)

Tendon	At Jacking		At Transfer	
	Pre-tensioning	Post-tensioning	Pre-tensioning	Post-tensioning
AFRP	$0.40f_{pu}$	$0.40f_{pu}$	$0.38f_{pu}$	$0.35f_{pu}$
CFRP	$0.65f_{pu}$	$0.65f_{pu}$	$0.60f_{pu}$	$0.60f_{pu}$
GFRP	not applicable	$0.55f_{pu}$	not applicable	$0.48f_{pu}$

3.3 Experimental Program for Strengthening of Concrete Members with External CFRP Cables

3.3.1 Objectives and Description of the Program

The main objectives of this experimental program are:

1. To investigate the behaviour of reinforced and prestressed concrete members strengthened with externally prestressed carbon fiber composites cables and to examine the effectiveness of this strengthening technique.
2. To study the effects of two parameters:
   - The member span-to-depth ratio (S/d_p)
   - The partial prestressing ratio (PPR)

   on the behaviour of the strengthened members.

In this investigation, a total of twelve partially prestressed concrete beams were tested. All the beams had the same rectangular cross section and were simply supported at the two ends when tested. The beams were divided into three groups; each group consisted of 4 beams of the same span-to-depth ratio but with different levels of prestressing. The beam span-to-depth ratio differs from one group to another. All the beams were subjected to two symmetrical concentrated loads applied in small increments at one third of the span length. In each group, three beams with different prestressing levels were subjected first to loads high enough to produce considerable cracking and deflections in each beam. The beams were then left under their own weight for a period of time to experience some time-dependent deformations before being strengthened with CFC external cables and tested up to failure. The fourth beam in each group was tested monotonically up to failure without strengthening and was used as control beam for comparison purposes.

In this series of tests, two main parameters were considered: (1) The span-to-depth ratio, S/d_p to investigate its effects on the change in stress in the externally prestressed cables and on their efficiency in strengthening damaged partially prestressed beams; (2) the level of internal prestressing expressed by the reinforcing index, ω and the partial prestressing ratio, PPR. For the purpose of this investigation, ω and *PPR* defined by the following equation were used:

$$\omega = \frac{A_{ps}f_{ps} + A_s f_y - A_s' f_y'}{b d_e f_c'} \tag{3.1}$$

$$PPR = \frac{A_{ps}f_{ps}}{A_{ps}f_{ps} + A_s f_y} \tag{3.2}$$

where

$$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y} \tag{3.3}$$

In the following sections, the experimental program and testing procedure are described in more details.

3.3.2 Geometry of Test Specimens

Details of the twelve simply supported beam specimens with a rectangular cross section of 150×280 for each of the three span lengths are listed in Table 3.2. The table gives information such as beam dimensions and span, beam designation, steel areas (tension and compression steel), types and numbers of internal and external prestressing tendons.

The twelve beams were grouped into three groups (B1, B2, and B3), each consisting of four beams, depending on the span-to-depth ratio:

Group 1: Beams B1 were of span L = 2000mm; S = 2250mm

Group 2: Beams B2 were of span L = 3500 mm; S =3750mm

Group 3: Beams B3 were of span L = 5000mm; S = 5250mm

where L is the span between the supports, and S the span between the end anchorages.

Table 3.2: Details of Test Specimens

Beam Dimensions	Beam Span	Beam Designation	Tension Steel A_s	Compression Steel A_s' (Plain Bar)	Internal Bonded Prestressing (7-wire strands)	External FRP Reinforcement CFCC
b = 150mm h = 280mm d_s = 250mm d_p = 210mm d_{pe} = 324mm (at midspan)	L = 2.0m S = 2.25 m S/d_p= 10.7	**RCB1**	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	0	2(1×7) strand 5.0 ϕ
		PPCB1-1 & PPCB1-1C	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	1×3/8in ($55.0mm^2$)	2(1×7) strand 5.0 ϕ only for **PPCB1-1**
		PPCB1-2	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	2×3/8in ($110mm^2$)	2(1×7) strand 7.5 ϕ
	L =3.5m S = 3.75 m S/d_p= 17.85	**RCB2**	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	0	2(1×7) strand 5.0 ϕ
		PPCB2-1 & PPCB2-1C	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	1×3/8in ($55.0mm^2$)	2(1×7) strand 5.0 ϕ only for **PPCB2-1**
		PPCB2-2	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	2×3/8in ($110mm^2$)	2(1×7) strand 7.5 ϕ
	L = 5.0m S = 5.25 m S/d_p= 25	**RCB3**	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	0	2(1×7) strand 5.0 ϕ
		PPCB3-1 & PPCB3-1C	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	1×3/8in ($55.0mm^2$)	2(1×7) strand 5.0 ϕ only for **PPCB3-1**
		PPCB3-2	2#10M ($200mm^2$)	2 (6mm) ($56.62mm^2$)	2×3/8in ($110mm^2$)	2(1×7) strand 7.5 ϕ

The tendon length/effective depth ratio *(S/d_p)* of the three groups were 10.7, 17.85, and 25, where d_p is the distance from the compression face of the beam to the centroid of prestressing steel. It is important to note that S and d_p are the main variables affecting the change in steel stress with the increase in load and not the span between the supports and the overall depth d. The span to overall depth ratios *(L/d)* of the three groups of beams were 9.5, 16.7, and 23.8, respectively.

The beams are designated as follows: RCB indicates reinforced concrete beam, with zero partial prestressing ratio ($PPR = 0$); PPCB means partially prestressed concrete beam. The first number after RCB or PPCB indicates the group number (Group 1 is for S/d_p = 10.7, etc.) and the second number (1 or 2) indicates the number of internal pretensioned steel tendons and hence the partial prestressing ratio in the beam. The control beams are designated as PPCB1-1C, PPCB2-1C and PPCB3-1C for groups 1, 2 and 3, respectively. They are identical to beams PPCB1-1, PPCB2-1 and PPCB3-1 without external prestressing. All beams were stored under laboratory conditions of 22 °C and 64% relative humidity for all the time before testing. Figure 3.1 shows a typical elevation, cross-section and external prestressing layout of a beam specimen.

For all externally prestressed beams, the CFCC were draped at midspan by means of a deviator (or a saddle). The depth from the top face of the beam to the bottom of the saddle is 324 mm. The eccentricity of the cables at the anchorages is zero. The length of cables used are 2920, 4410 and 5910 mm for beam span lengths S= 2250, 3750, and 5250 mm, respectively. These lengths of cables include the length required for anchorage.

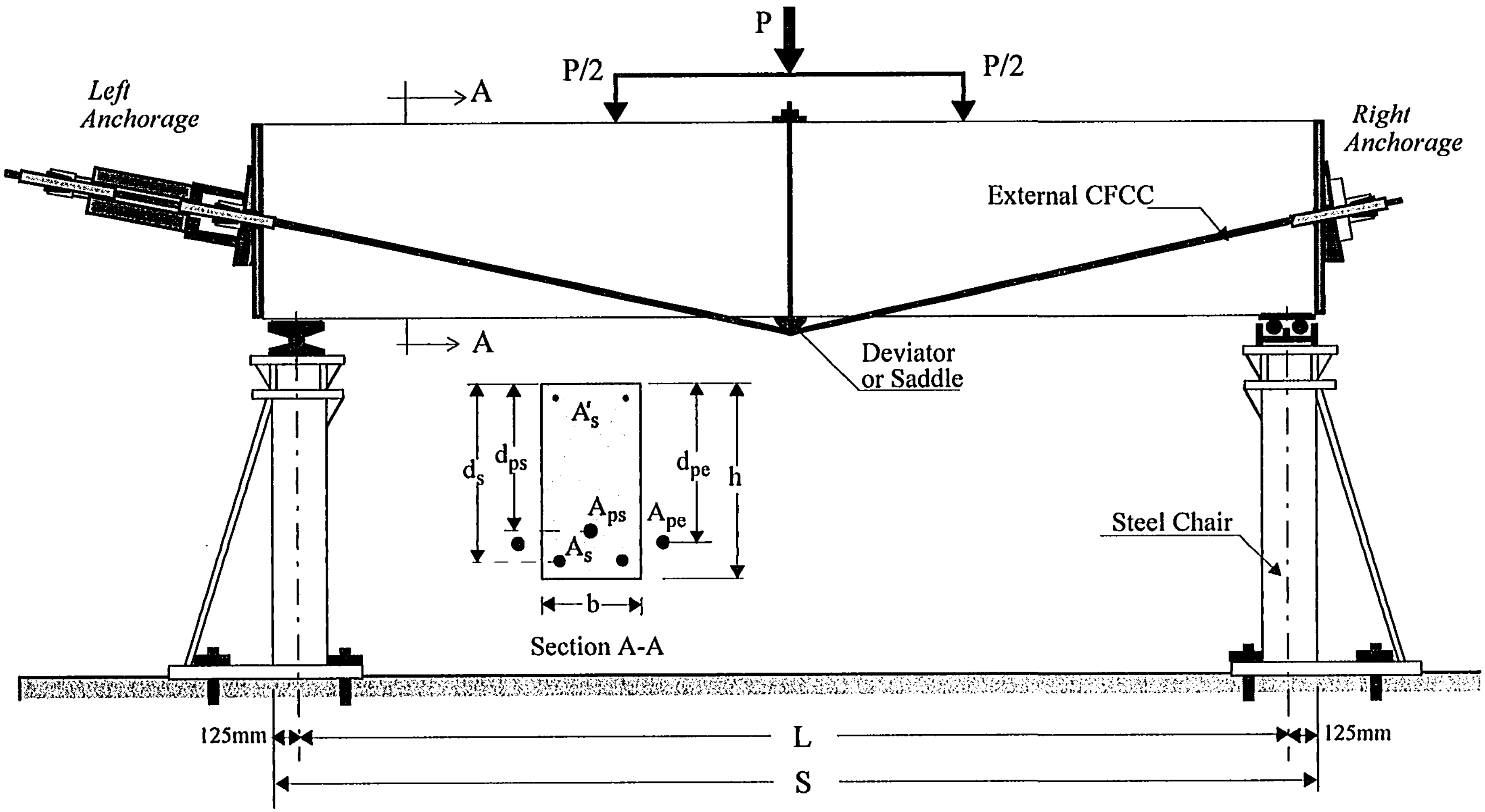


Figure 3.1 Layout of Test Specimen

3.3.3 Design of Concrete Beams

The design of the concrete beams was carried out according to the Canadian Code (CSA/CAN3-A23.3-94). The area of internal steel reinforcement (nonprestressed and/or prestressed) was selected to provide ultimate strength close to the lower limit of under-reinforced beams. This allowed the external prestressing to be added without over-reinforcing the beam (which would lead to premature brittle failure of the concrete in compression). Each beam was designed with sufficient reinforcement to carry its own weight without cracking. Table 3.3 shows the requirement of the code in terms of the reinforcement ratio and index to ensure a ductile behaviour of all beam of Group 1. Table 3.4 gives the cross-sectional areas and the yield strength of the ordinary reinforcing steel (tension and compression) the areas and ultimate strength of internal bonded prestressing steel, and the combined reinforcing index and the partial prestressing ratios for beams of Group 1. Beams in Groups 2 and 3 have the same data.

The material properties, fabrication technique, prestressing system, instrumentation, test setup and procedure are described in the following paragraphs.

3.4 Beam Fabrication

3.4.1 Preparation of Formwork

The specimen formwork was manufactured from 12.5 mm thick wood plates. It was fabricated in order to cast three beams at the same time as shown in Figure 3.2, and in a way to facilitate its stripping and reassembling. Holes were drilled in the ends of the formwork at premeasured locations for accurate placement of the prestressed tendons. The formwork was braced at the top in order to prevent any lateral movement or change in dimensions during casting. The inner sides of the wood plates were coated by a thin layer of wax oil to facilitate their removal and cleaning.

Table 3.3: Code Requirements for Reinforcement Ratios and Index of Beams in Group 1*

Beam Designation	ρ_s	ρ_s'	ρ_{ps}	ω_s	ω_s'	ω_{ps}	ω
RCB1			0			0	0.049
PPCB1-1	0.0053	0.0015	0.0017	0.0609	0.0119	0.0924	0.142
PPCB1-2			0.0035			0.1849	0.234

for Reinforced Concrete section:

$\rho_{min} = 1.4/f_y = 0.0035\%$

$$\rho_{max} = \frac{0.85 f_c' \beta_1}{f_y}\left(\frac{600}{600 + f_y}\right) = 0.036\%$$

for Partially Prestressed section:

$\bar{\omega}_{min} \cong 0.033$

$\bar{\omega}_{max} = 0.36\beta_1 = 036(0.81) = 0.2916$

*same data for beams in Groups 2 and 3.

Table 3.4: Reinforcement Properties of Beams in Group 1*

Beam Designation	Tension Steel		Compression Steel		Internal Bonded Prestressing		Combined Reinforcing Ratio	Partial Prestressing Ratio
	A_s (mm^2)	f_y (N/mm^2)	A_s' (mm^2)	f_y' (N/mm^2)	A_{ps} (mm^2)	f_{pu} (N/mm^2)	ω	*PPR*
RCB1					0	0	0.05	0
PPCB1-1	200	400	57	275	55.0	1860	0.142	0.53
PPCB1-2					110	1860	0.234	0.70

*same data for beams in Groups 2 and 3.

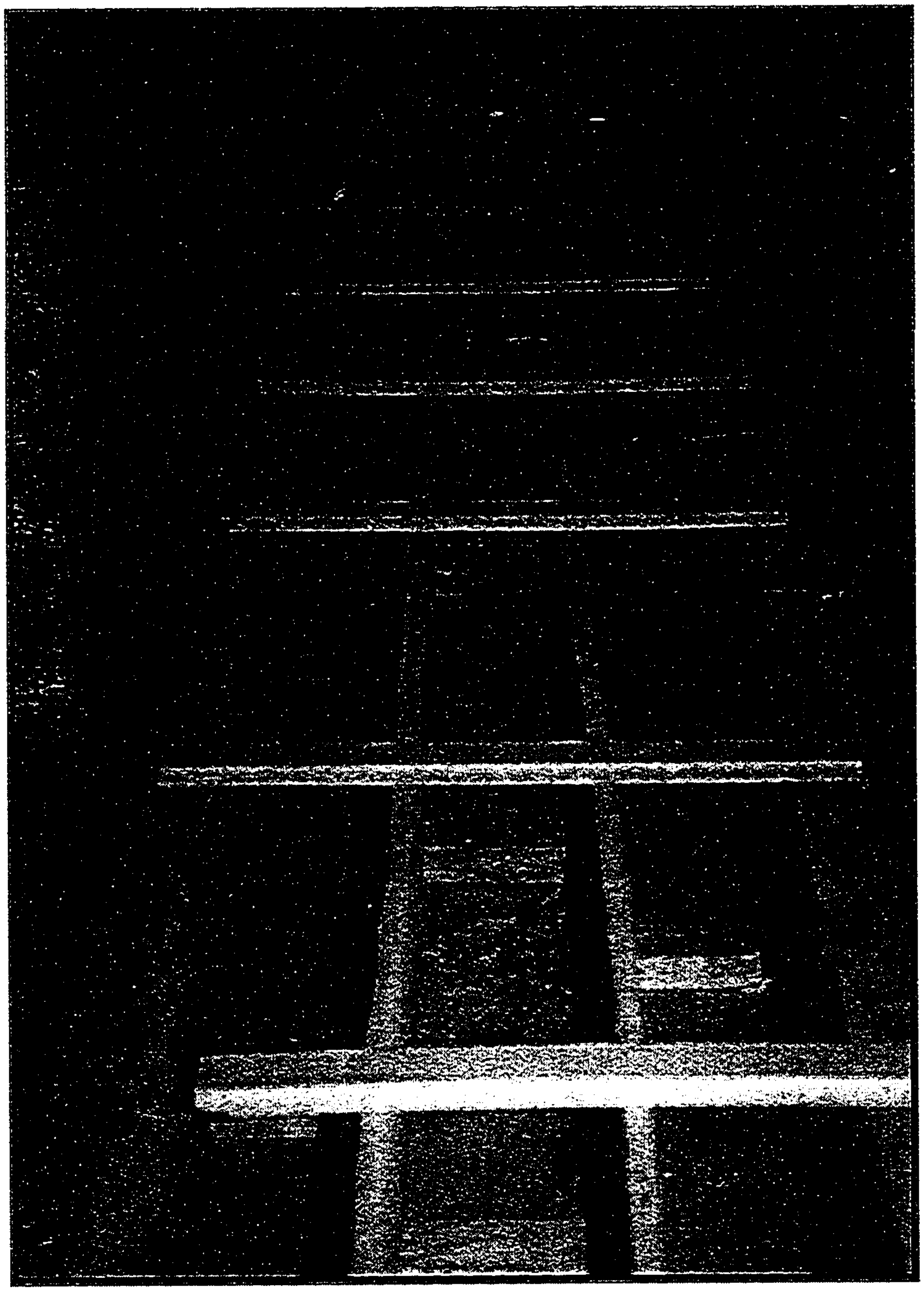


Figure 3.2 Specimen Formwork

3.4.2 Preparation of Reinforcing Cages

The reinforcing bars were cut in the laboratory to the required length. In order to measure the strain variation in the reinforcing bars, strain gauges were installed at the middle of the tension reinforcement. Prior to casting the beams, strain gauges were attached to the flexural reinforcement as shown in Figure 3.3. A 10 mm long strain gauge was attached at the midspan of each of the #10M tensile reinforcing bars, the ribs in these locations were removed, and the strain gauges were attached using epoxy resin. The gauges were covered with silicon, and the wires, which had been soldered to the strain gauges, were placed along the longitudinal reinforcement and they projected from the beam in one location. The shear reinforcement was cut and bent using a special bender to form closed type stirrups and then tied to the longitudinal top and bottom reinforcement. The spacing between stirrups was kept constant along the length of the beam at 200 mm. The reinforcing cage is shown in Figure 3.4 placed inside the oil coated wood formwork. The longitudinal bottom and top nonprestressed reinforcements were identical for all beam specimens. The bottom reinforcement was kept at its proper elevation using small chairs (25.4 mm high) while the top reinforcement was tied to the transverse reinforcement at the desired elevation.

3.4.3 Prestressing Procedure

The prestressing bed and the support system used to jack the prestressing reinforcement for the prestressed beams is shown in Figure 3.5. The internal bonded tendons were pretensioned inside the formwork before casting the concrete. The prestressing forces were applied using hydraulic jack with 12 tons capacity. Although the initial prestressing force required in each tendon was 60% of its ultimate tensile strength,

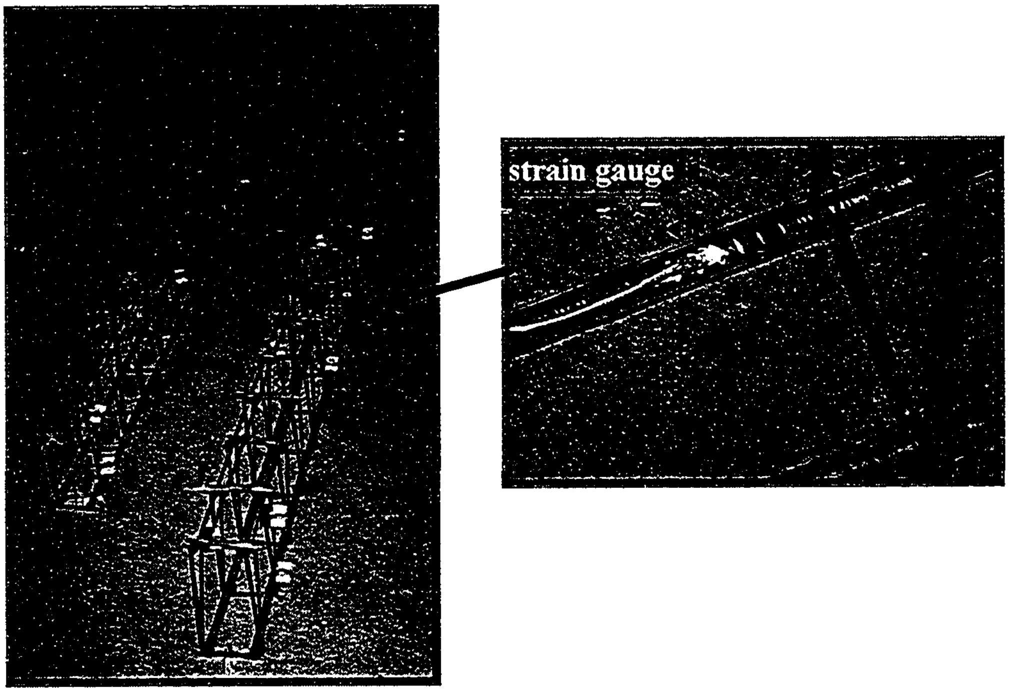


Figure 3.3 Steel Strain Gauge

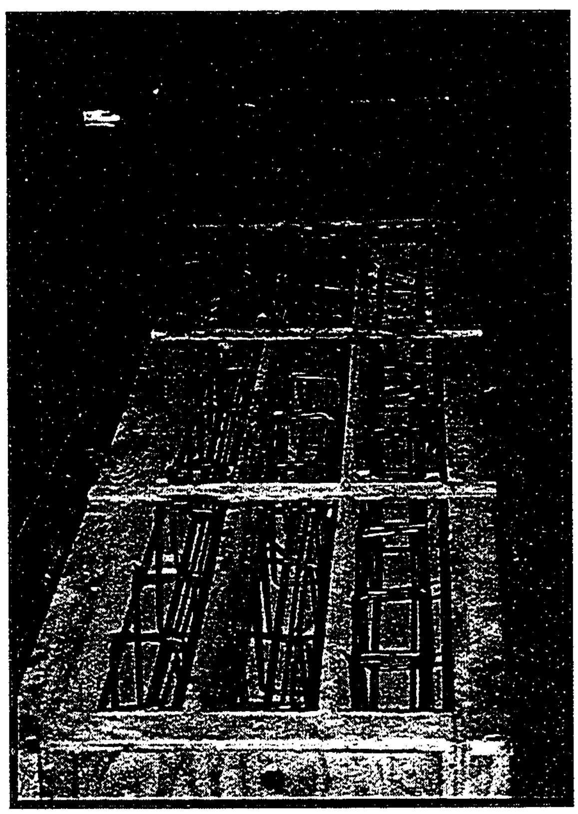
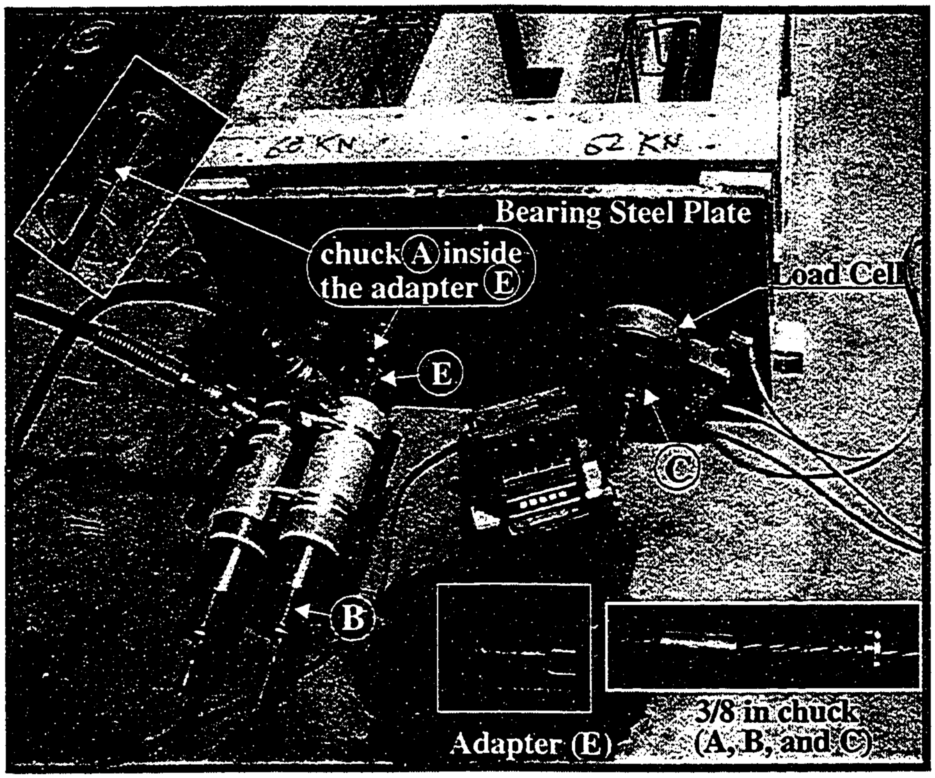


Figure 3.4 The Reinforcing Cage Placed Inside the Oil Coated Wood Formwork



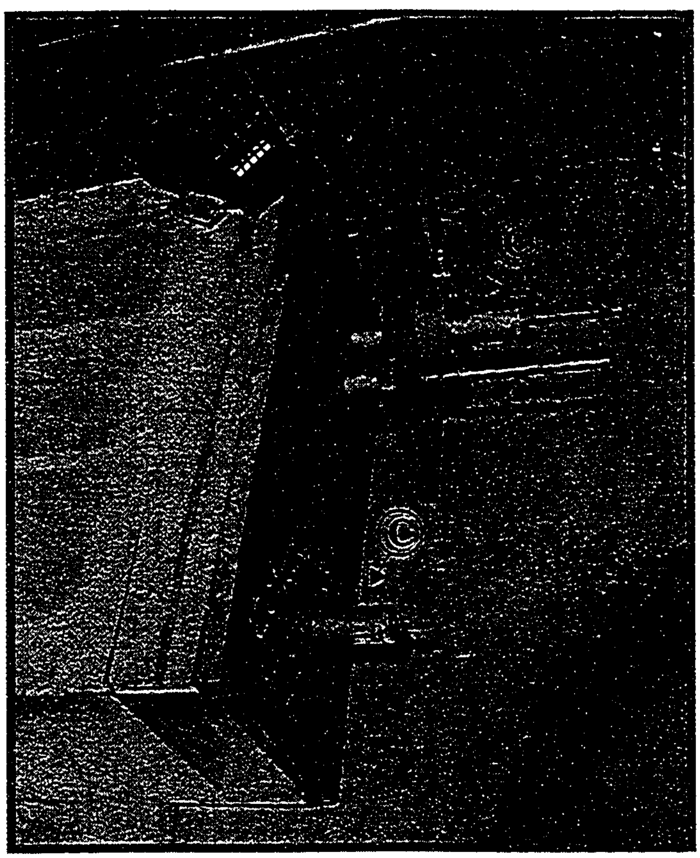


Figure 3.5 Prestressing Bed

f_{pu}, the jacking force applied to the tendon was equal to 0.65 f_{pu} to compensate for anchorage set losses at the time of prestressing and for the steel relaxation losses in the period between the application of prestressing and the casting of concrete. The anchorages used to hold the prestressed strands were PSI Super Multiple use standard chucks 3/8in supplied by Prestress Supply Inc. FL. USA, made for gripping 3/8in diameter 7-wires prestressing cables. The chucks at both anchored ends were supported directly on a steel bearing plate attached to the formwork ends in order to ensure that the bearing stresses are under the allowable limits. Three chucks (A), (B), and (C) were used for each prestressed strand.

The prestressed strand was held by the anchorages (B) and (C) at its ends. The anchorage (C) at the far end was bearing against a load cell, so that the force in the prestressed wire could be monitored. At the jacking end, the pressure by the hydraulic jack was increased gradually until the desired prestressing force was reached. The pressure was then held constant. The ram of the hydraulic jack was pressing against an adapter [a hollow cylinder (E)] which was in turn bearing against the end steel plate, and containing inside it the anchorage (A). When the required prestressing force was reached, the anchorage (A) was gripped. The prestressed strand was then held by the anchorages (A) and (C). The hydraulic jack, the adapter, and the anchorage (B) were removed immediately after the prestressing operation was completed.

The prestressing force was monitored by the calibrated load cells placed at the end of each tendon. Just before casting of the concrete, additional prestressing forces were added when the stress in the tendons decreased below $0.6f_{pu}$. Three days after casting the concrete, the prestressing forces were released.

3.4.4 Casting of Concrete

Three beams of equal spans (i.e. of each group) were cast at the same time. The last casting operation included the control beams, one from each group. A standard concrete mix was used, with a minimum aggregate size of 1/4 inch and a minimum compressive strength of 35 MPa. A Mikasa Concrete Vibrator 13/4 in. DY.22.WK.75.MTH.180 was used for internal vibration of the concrete mix during casting as shown in Figure 3.6. Care was taken not to damage the strain gauges installed on the reinforcing bars during casting. The concrete surface was then levelled, and covered with burlap sheets. Forms were removed seven days after casting. The resistance of the strain gauges attached to the reinforcing bars were checked after removing the formwork for any possible damage during casting.

Following casting the beam, control cylinders were cast from the same mix. The concrete compressive strength tests were performed on three samples of 3×6 in. (75×150 mm) standard concrete cylinder taken for each beam specimen to determine the concrete compressive strength at the time of testing.

3.5 Material Properties

3.5.1 Concrete

The concrete used for casting the beams was provided by a local supplier (Beton Mobile du Quebec) and the beams were cast 24 hours after jacking. The target strength of the ready mix concrete was 35 MPa. Three concrete cylinders were cast from each patch for each beam and were tested in compression at the same day of testing. The measured compressive strength of the concrete cylinders at the time of testing (stage I: before

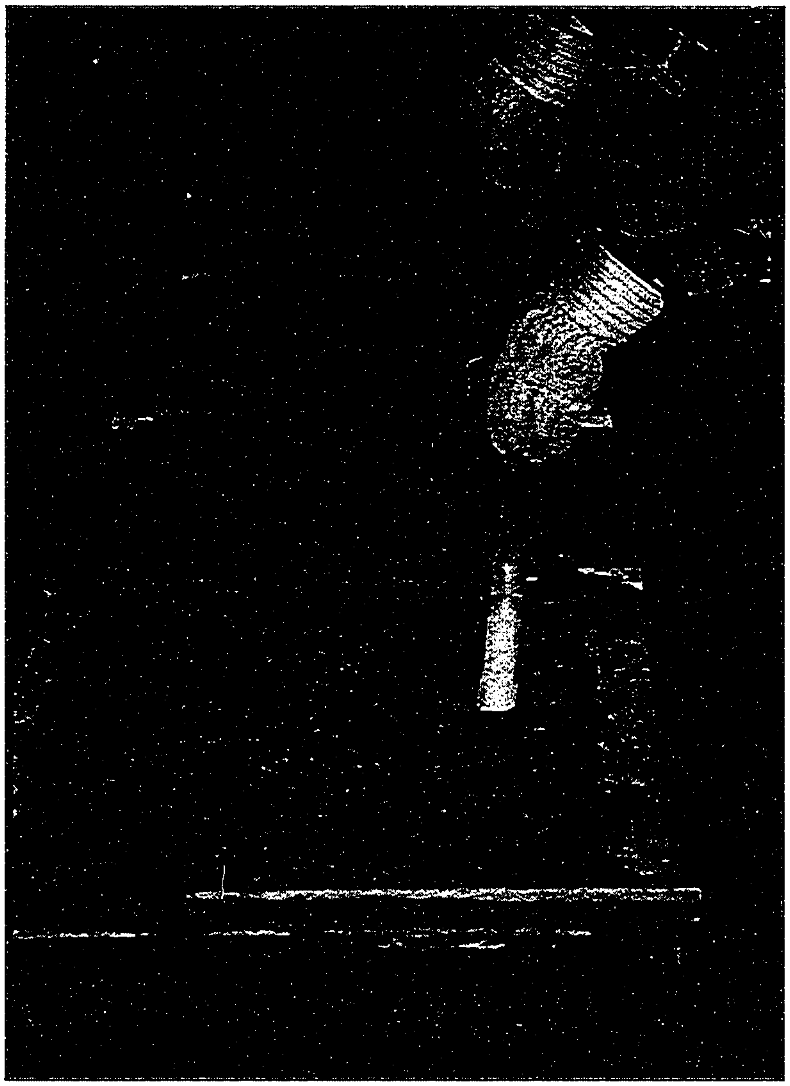


Figure 3.6 Casting of Concrete

Table 3.5: Concrete Strength for Beam Specimens at the Time of Testing (Stage I) (before Strengthening)

Beam Type	Concrete Strength at Testing				Age at Testing (day)
	Cylinder No. 1	Cylinder No. 2	Cylinder No. 3	Average f_c' (MPa)	
RCB1	44.70	44.50	NT	44.60	50
PPCB1-1	42.17	41.97	44.90	43.01	13
PPCB1-1C	37.90	35.86	38.01	37.26	28
PPCB1-2	49.78	45.88	NT	47.83	61
RCB2	41.90	42.90	NT	42.40	44
PPCB2-1	42.90	46.30	43.4	44.20	22
PPCB2-1C	37.10	37.00	36.00	36.70	14
PPCB2-2	46.40	53.70	47.40	49.20	24
RCB3	35.40	36.20	37.10	36.20	34
PPCB3-1	47.40	52.50	47.60	49.20	70
PPCB3-1C	35.14	38.07	35.44	36.22	15
PPCB3-2	42.30	42.70	NT	42.50	38

The target strength of concrete was 35 MPa
The area of the cylinder = 4560 mm^2
NT indicates not tested

strengthening) are shown in Table 3.5. The concrete strength values given in Table 3.5 include the average compressive strength of three cylinders taken for each beam specimen tested. The concrete mix proportions by weight were 1 cement: 2.05 coarse aggregate: 1.72 fine aggregate. No superplastisizer was used to increase the concrete workability. The water/cement ratio was 0.37 and the cement content was 450 kg/m^3. High early strength concrete type 30 cement and uncruched well rounded graded river gravel with maximum aggregate size of 1/4 in were used.

3.5.2 Reinforcing Steel

The flexural reinforcement used as nonprestressed tension (bottom) reinforcement consisted of deformed steel bars of size 10M of nominal diameter equal to 11.3mm, and area of each bar equal to 100 mm^2, with minimum yield strength of 400MPa (Grade 60). In addition, all beams contained 2-6mm plain bars of 275 MPa (Grade 40) yield strength as top reinforcement in order to support the stirrups. It should be mentioned that this top reinforcement was considered when calculating the reinforcing index. The modulus of elasticity of the bars was estimated at 200 GPa.

3.5.3 Prestressed Reinforcement

3.5.3.1 Internal Bonded Prestressing

Low relaxation seven-wire strands, Grade 270 with ultimate strength of 1860 MPa were used as internal bonded prestressed reinforcement. The strands have a nominal diameter of 9.525 mm (3/8 in.), and a cross-sectional area of 54.8 mm^2 (0.085 in^2.), and a modulus of elasticity of 200 GPa. The strands consisted of seven individual steel wires and were made by tightly wrapping six of the wires around the seventh.

3.5.3.2 External Prestressing

Nine beams were strengthened with externally prestressed Carbon Fiber Composite Cables (CFCC) consisting of 1×7 strands of nominal diameter of 5mm or 7.5mm, and modulus of elasticity of about 137 GPa. Figure 3.7 shows the CFCC cables used in this study. Some technical parameters, shown in Table 3.6, are available from the supplier.

The tendons were produced and supplied by Tokyo Rope Manufacturing Co. Ltd. and Toho Rayon Co. Ltd. in Japan. The CFC cables are made of carbon fiber yarns twisted together, similar to the 7-wire steel tendons widely used in the prestressed concrete industry. According to the manufactures CFCC is a kind of CFRP as well as a kind of cable. Therefore the CFCC combines the features of both. Since it is kind of CFRP, it demonstrates such features as "strong", "lightweight", "strongly resists salt damage", "nonmagnetic", and "little relaxation loss". Since it is a kind of cable, it is "flexible", "easily usable for making a variety of products" and possesses "very good adhesion to concrete". "Flexibility" makes it possible to demonstrate such superior features as easy reeling and transportation, possibility of producing long cable, easy bending and handling.

In the following section, the main characteristics of CFCC will be presented and precautions related to handling and prestressing the cables will be discussed.

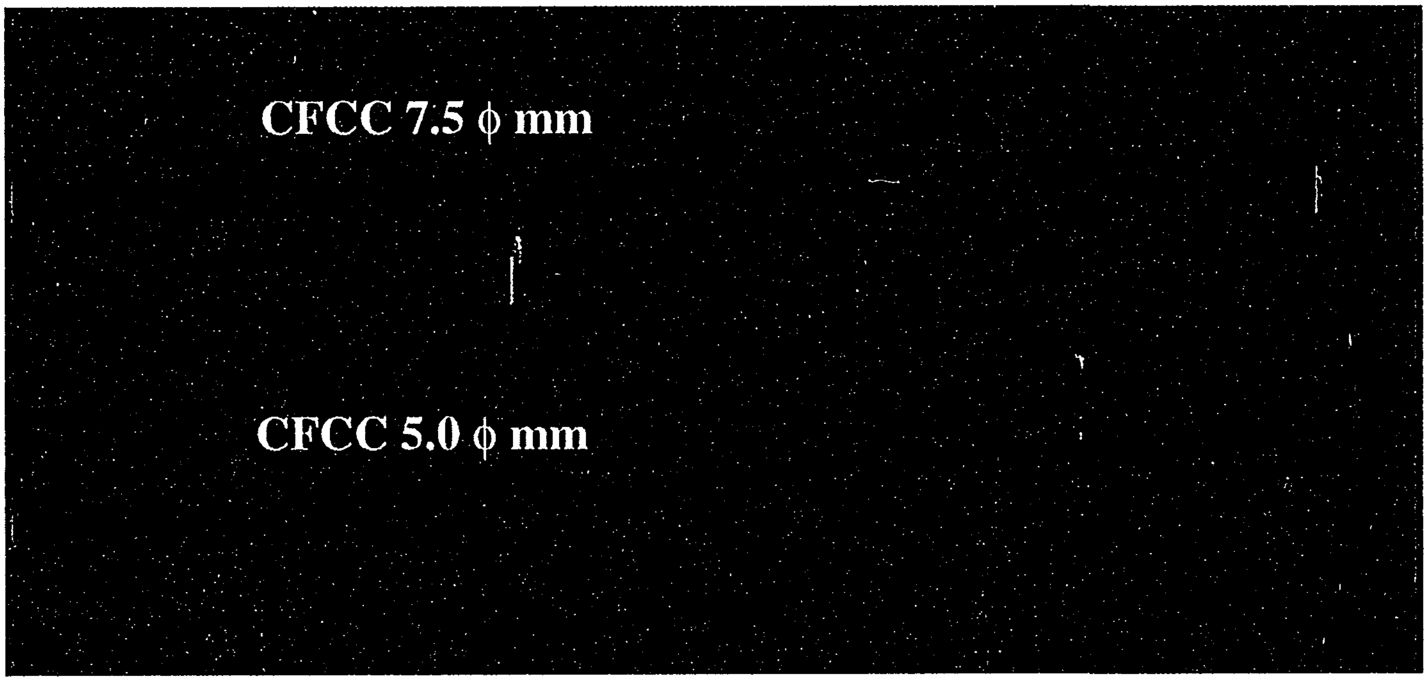
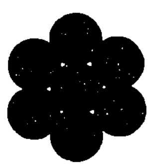


Figure 3.7 CFCC Cables

Table 3.6: Standard Specifications of CFCC (Tokyo Rope 1993)

<table>
<tr><td rowspan="10">
Cross-section
1×7-strand
CFCC</td><td>Diameter (mm)</td><td>5.0 ϕ</td><td>7.5 ϕ</td></tr>
<tr><td>Cross-section</td><td>7-strand</td><td>7-strand</td></tr>
<tr><td>Effective Cross-sectional Area (mm^2) Including the Area of Resin</td><td>10.1</td><td>30.4</td></tr>
<tr><td>Guaranteed Breaking Load (kN)</td><td>18</td><td>57</td></tr>
<tr><td>Nominal Tensile Strength of Cable (kN/mm^2)</td><td>2.12</td><td>2.04</td></tr>
<tr><td>Breaking Load (kN)</td><td>21.4*</td><td>60.8**</td></tr>
<tr><td>Nominal Modulus of Elasticity (kN/mm^2)</td><td>144</td><td>135</td></tr>
<tr><td>Standard Mass (g/m)</td><td>24</td><td>64</td></tr>
<tr><td>Heat Resistance Grade</td><td>130°C type</td><td>130°C type</td></tr>
<tr><td>Elongation at Failure</td><td>1.6%</td><td>1.4%</td></tr>
<tr><td colspan="4">* number of specimens tested = 6 (Max = 23.3 kN, Min = 20.2 kN)
** number of specimens tested = 3 (Max = 64.3 kN, Min = 57.2 kN)</td></tr>
</table>

3.6 CFCC characteristics

The type of carbon fibers used in this study is a unidirectional Carbon Fiber Composite Cable (CFCC) consisting of continuous high strength carbon fibers embedded in a polymer matrix. These fibers are oriented along the cable axis in order to achieve maximum strength and stiffness in the longitudinal direction. The individual wires of CFCC are manufactured by a roving prepreg process using carbon fibers of polyacrylonitrile (PAN base carbon fiber BESFIGHT HTA) type supplied by Toho Rayon and an epoxy resin. The fiber content is 60 percent by volume. The prepreg is twisted to create a fiber core which is then wrapped circumferentially by a thin layer of polymeric synthetic yarns. The purpose of the yarn is to protect the fibers from the ultra-violet radiation, to protect the carbon fibers against damage and mechanical abrasion during use, and to improve the bond properties of the wire to concrete. Furthermore, the spiral grooves on the outside of the cable helps to anchor the cable in concrete or in casting of terminal fixers.

As an FRP material, CFCC possess attractive qualities such as high tensile strength, light weight, high bond strength with concrete, excellent corrosion resistance, lower relaxation than steel tendons, and non-magnetic properties. As cables, CFCC have excellent flexibility, and are very easy to handle.

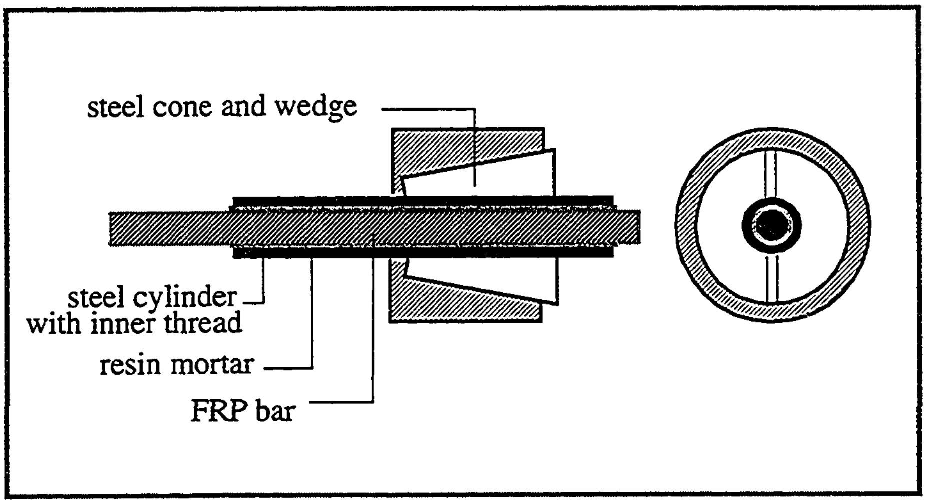
Santoh et al. (1993) conducted relaxation tests on CFCC and steel strands at room temperature. They found that at 0.5 P_u the relaxation value after 100 hours for CFCC was 0.48% and for steel was 1.02%; at 0.65 P_u the relaxation of CFCC was 0.81% while that of steel was 2.28%. They conclude from their tests that the relaxation of CFCC is 1/2 or less that of steel. Further, CFCC have great ability to be cut with a saw and can be easily fastened with nails or screws. The bending stiffness of CFCC allows sufficient flexibility for easy coiling, handling, and installation. The cables (or ropes) are made from either single, seven, nineteen or thirty-seven wire strands that are twisted to allow better stress distribution through the cross-section. The single core cable can be of 3.0 - 5.0 mm diameter. The multi core cables can be of 5.0 - 40.0 mm diameter. The breaking load of CFCC can vary from 1.43kN to 1100 kN. The CFC cables have a tensile strength of 1.8 GPa, a modulus of elasticity of 137 GPa, and a typical maximum elongation at failure of 1.6%. The bond strength is 4.2 MPa. According to the manufacturer, the variation of elastic tensile modulus of CFCC is quite minimal and falls in the range of 200 to 230 MPa. The lateral compressive strength of CFCC is low.

Because the linear expansion coefficient of CFCC 0.6×10^{-6} m/m/°C, which is extremely small compared to the linear expansion coefficient of concrete (10.0×10^{-6} m/m/°C), there is concern that the adhesion between the concrete and the CFCC may be destroyed due to temperature fluctuations. Santoh et al. (1993) conducted temperature cyclic tests by attaching a distortion gauge to CFCC in a prestressed concrete beam, and measuring the distortion changes at 20°C for each of several temperature cycles. Even when repeating fluctuations from -10 to 40°C for 50 times in a 24 hour period, they found that there was no distortion change arising because of the reduction of the of adhesive strength at 80 cm or more inside from the ends of the beam.

CFCC can be manufactured into very long cables. The maximum length of CFCC now available is 600 m, with future plans to make lengths up to 3000 m. In their unique construction these cables utilize about 90% of the carbon fiber strength and 97% of the carbon fiber tensile modulus, and therefore can be flexible enough to be coiled on drums of reasonable diameter. CFCC are mainly used as prestressing tendons and reinforcement. They can also be used as supporting cables in cable-stayed and suspension bridges.

3.6.1 Anchoring System for CFCC

Many anchoring devices have been developed for ordinary prestressing systems. These devices can not be applied directly to FRP tendons since their surface is very delicate compared to that of prestressing steel. The selection of anchorage type for CFCC depends on the size of the cable and the nature of the application. In typical prestressing processes steel wedges are used to clamp the individual strands to the anchor plates. Steel cables can be clamped directly with wedges whereas CFCC need to be protected at their ends first with a surrounding steel tube before they can be gripped with wedges. This is known as "the metal die-cast wedge system" and is shown in Figure 3.8. In this system, a special alloy is molten and die-molded on the end portion of the CFCC, a steel pipe is installed onto the die-cast portion and pressed to integrate with the CFCC. With this system, the wedge can be installed through one-touch action and the stressing force can be immediately introduced. The application of die-cast method can be also performed at the cable manufacture's shop; in this case, the precise tendon length needs to be determined in advance. In this anchoring method, the wedges can be reused, and in fact, the special alloy can also be reused after melting. Thus, this system is superior in flexibility and workability. For all anchorage systems, steel wedges are used to clamp individual cables to an anchorage head, similar to the method for prestressing steel cables.



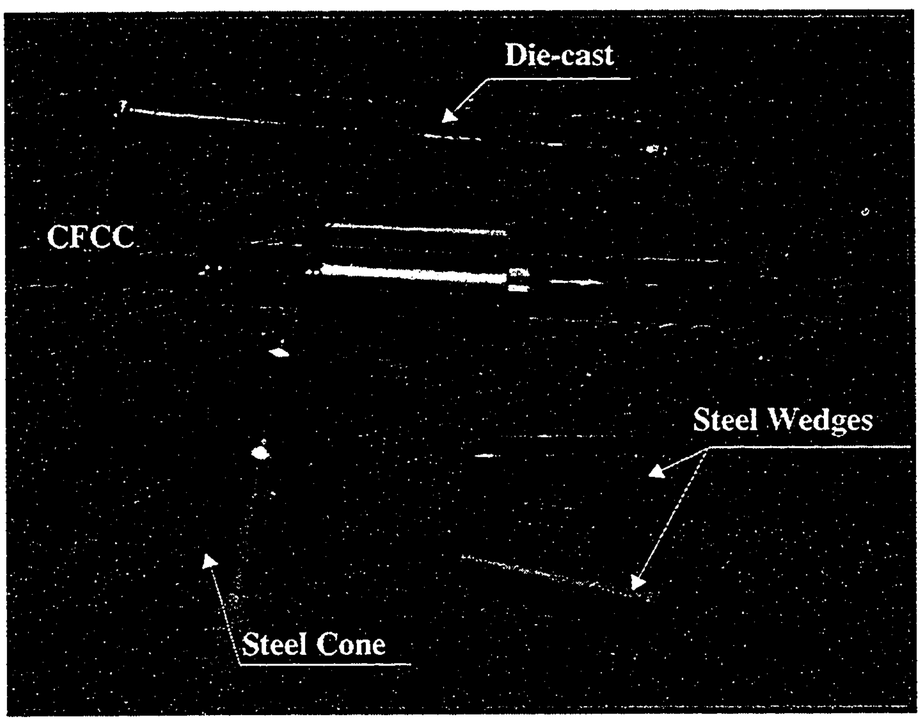


Figure 3.8 Wedge System with Die-Casting

3.6.2 Precautions for Handling and Arrangements for Prestressing and Anchoring of CFCC

The CFCC are made of non-metallic composite material consisting of carbon fibers and resin. The CFCC have many properties different from metallic materials. They are sensitive to wearness, impact, shear strength, heat and flame. Therefore, they must be handled with care. The following important instructions were provided by the manufacture concerning handling, installation and prestressing of CFCC for prestressed applications:

3.6.2.1 Handling of CFCC in General

1. When removing a cable from the container or removing a fixture, the terminals of a cable must be held tightly by hand to avoid jumping out of CFCC. The terminals of the CFCC must be removed from the container one at a time.
2. Since the 5.0mm and 7.5mm diameter CFCC are very fine, extreme bending must be avoided.
3. The minimum bending diameter is approximately 80 times the diameter of CFCC. However, bending a portion of CFCC within 300 mm from the mouth of die-cast is absolutely forbidden (see Figure 3.9). This may lower the breaking strength of the cables by shearing damage.
4. In case of using double die-cast at one end of the cable, bending between the die-casts must be avoided since such parts can be easily damaged with high possibility of low-load breakage.
5. During installation the following must be prevented:
   - Rubbing the ground when pulling out CFCC.
   - Bending and damage by sharp edges of guide equipment or tools.

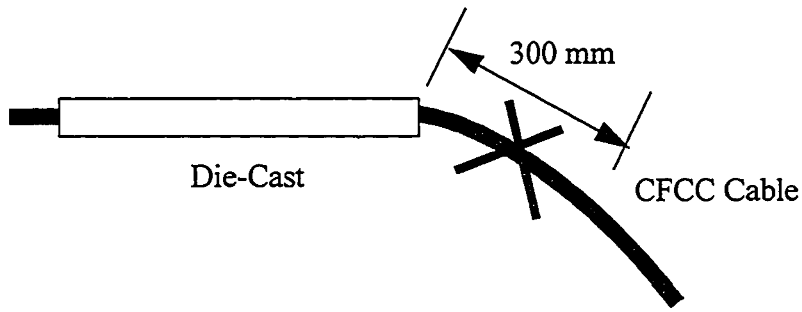


Figure 3.9 Bending the Cable Close to the Die-Cast must be Avoided

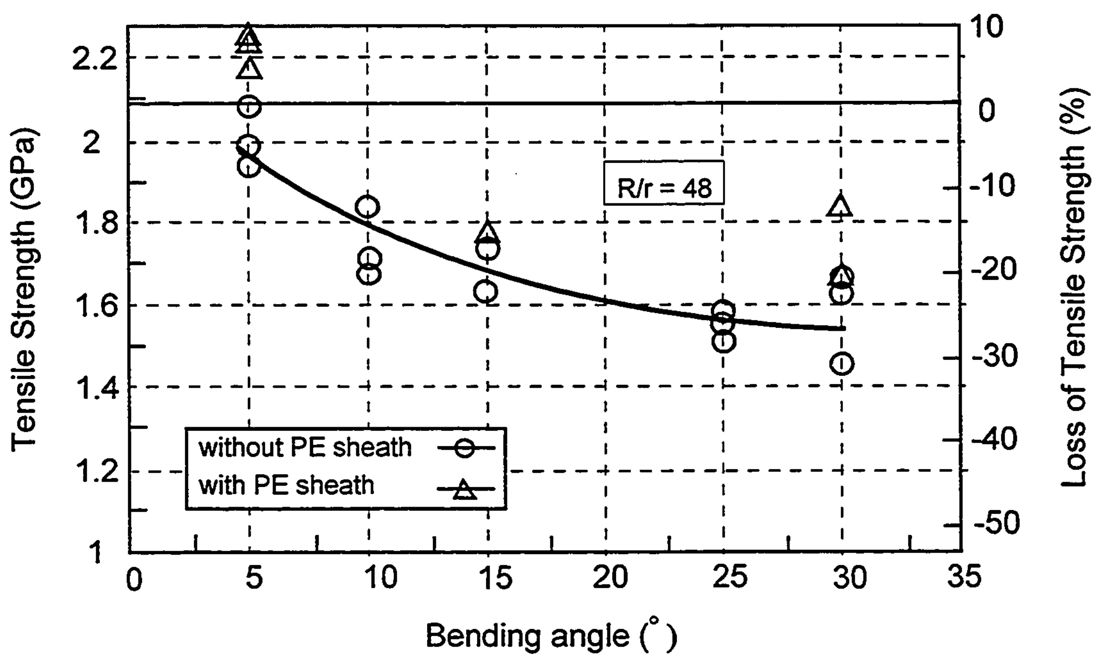


Figure 3.10 Relationship Between the Tensile Strength and Bending Angle with and without Sheath (after Santoh, 1993)

6. CFCC must be protected from any showers of sparks such as weld, cutting with gas or grinder.
7. Tension must not be applied to the CFCC which are bent, twisted or hung at contact portions with guide equipment, tools or star wraps.
8. Too high temperatures in a period of steam heating must be avoided, because the resin strength in the terminal sockets of the CFCC is decreased rapidly under temperature higher than 50 $^{\circ}$C.
9. Cutting the CFCC should be done with grinder-cutter or fretsaw; clipper and gas melt cutting must not be used.

3.6.2.2 The Deviator System

One important point which should be taken into consideration when using CFCC as pretensioned or externally prestressed draped tendons, is the effect of bending at the draped points on the ultimate strength of the cables. Figure 3.10, adopted from the Tokyo Rope catalogue, depicts the relationship between the tensile strength of the CFCC and the angle of curvature when the cables are bent while subjected to tension. As can be seen, a slight reduction in the cable strength is observed when the angle of curvature is 10° or less. Thus this reduction in strength of the CFCC can be mitigated by flaring the cables at the draped points. This was achieved for the beams tested in this investigation by the use of the deviator (saddle) system shown in Figure 3.11. The deviator consisted of a 40 mm thick part of a solid steel cylinder of 100 mm diameter. The steel part was 254 mm long and attached to the bottom of the beam at midspan by means of the steel rods shown in Figure 3.11. The hooks shown in the figure attached to the saddle ensure the lateral location of the cables with respect to the beam at midspan

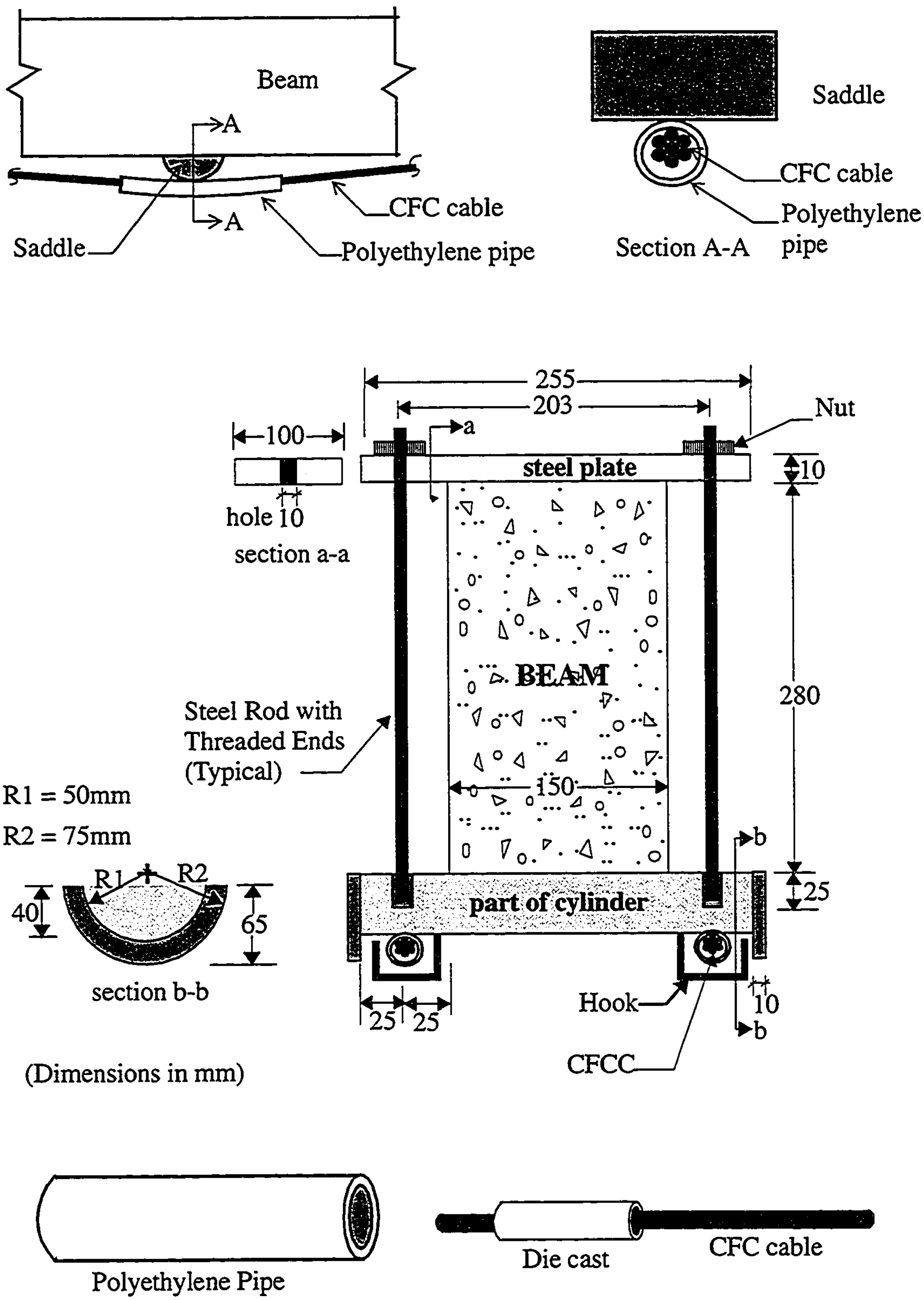
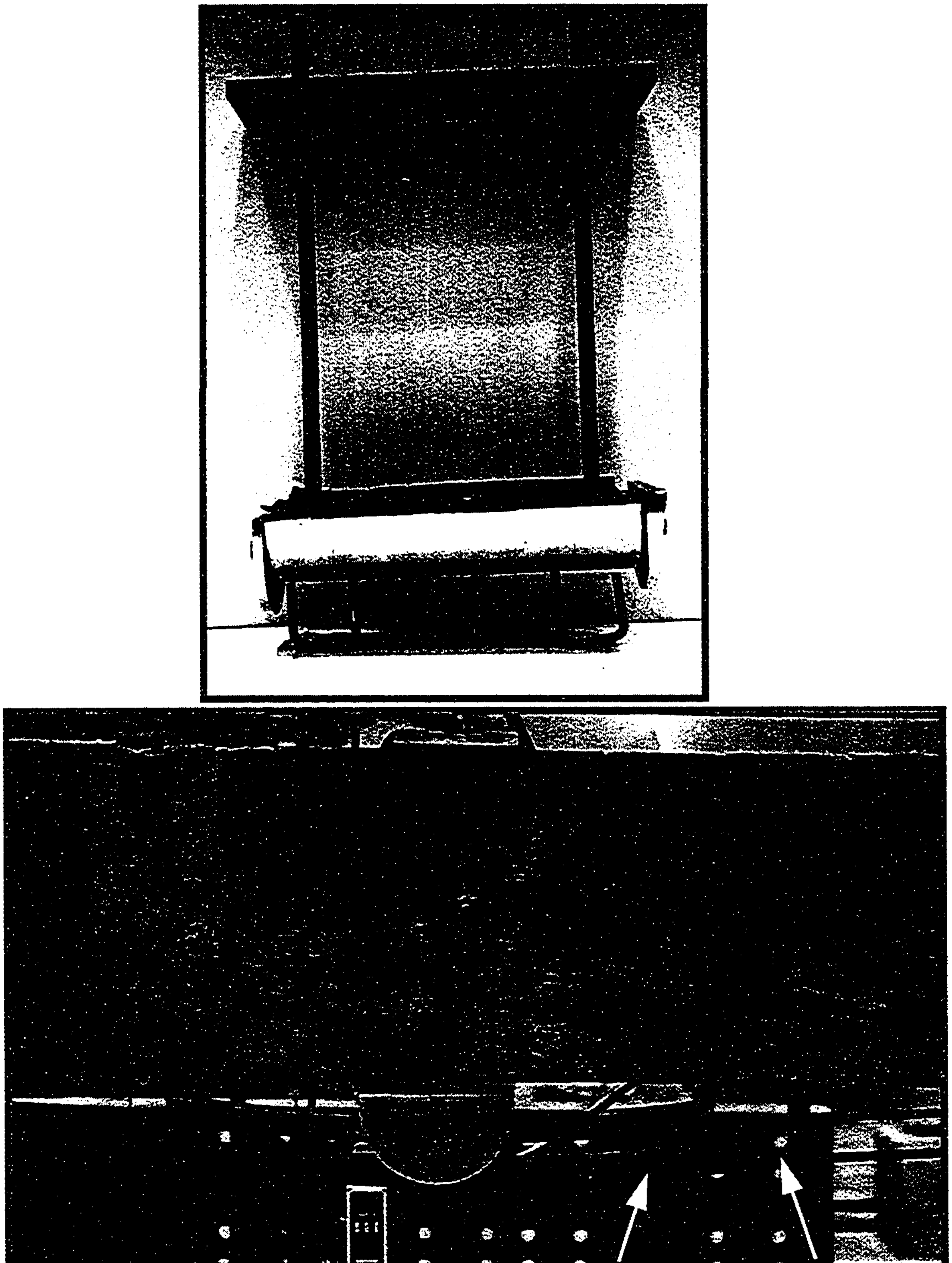


Figure 3.11 The Deviator System

In order to reduce the friction between the CFCC and the saddle, a cushion material or teflon sheaths must be used at the saddle point. According to the manufacturer, this is particularly important when the angle of bent at the saddle is larger than 5 degrees. Friction in this case may cause great reduction in the CFCC strength and may lead to breakage of the cables. The cushion material used in the present investigation was in the form of low density Polyethylene (PE) pipes of 27 mm outer diameter, 19 mm inner diameter, 4 mm thickness and 500 mm length. One PE pipe was provided for each CFC cable. The following precautions and steps were followed when the CFCC and die-cast were inserted into the PE pipe and the cables were placed under the saddle:

1. Before inserting the CFCC and the die-cast into the PE pipe, the pipe was made straight.
2. Inserting the CFCC and the die cast into the pipe was handled with great care, because if the die-cast was stuck inside the pipe and was forced into the pipe, the CFCC could be broken or damaged.
3. After the CFCC and the die-cast were inserted into the pipe, the PE pipe was placed centered inside the hook under the saddle part as shown in Figure 3.11. Photographs of the deviator before installation and during testing are shown in Figure 3.12.



Polyethylene Pipe

CFCC

Figure 3.12 The Deviator Before Installation and During Testing

3.6.2.3 Jacking and Anchoring

1. During tensioning the CFCC devices such as centre hole jacks, the cylinder of the jack often rotates relative to the jack causing twisting and unwinding of CFCC. This unwinding causes a drastic reduction in strength of the CFCC. In order to avoid twisting and unwinding of the CFCC, special arrangement has been made in the present experiments as shown in Figure 3.13 to prevent rotation of the cylinder during prestressing and anchoring.

2. Contact of the mouth of die-cast with the cylinder of the jack or the bearing plate must be avoided. Such contact may cause bending or crushing of CFCC at the mouth of the die-cast.

3. In order to prevent slippage of the wedge, the die-cast must be gripped with the wedge leaving at least 10 mm from the end of the die-cast.

4. As in the case of steel strands, more than half of the wedge length should eventually be put into the sleeve (the female cone). To facilitate insertion of the wedge into the sleeve, a thin layer of lubricant should be applied to the outer surface of the wedge.

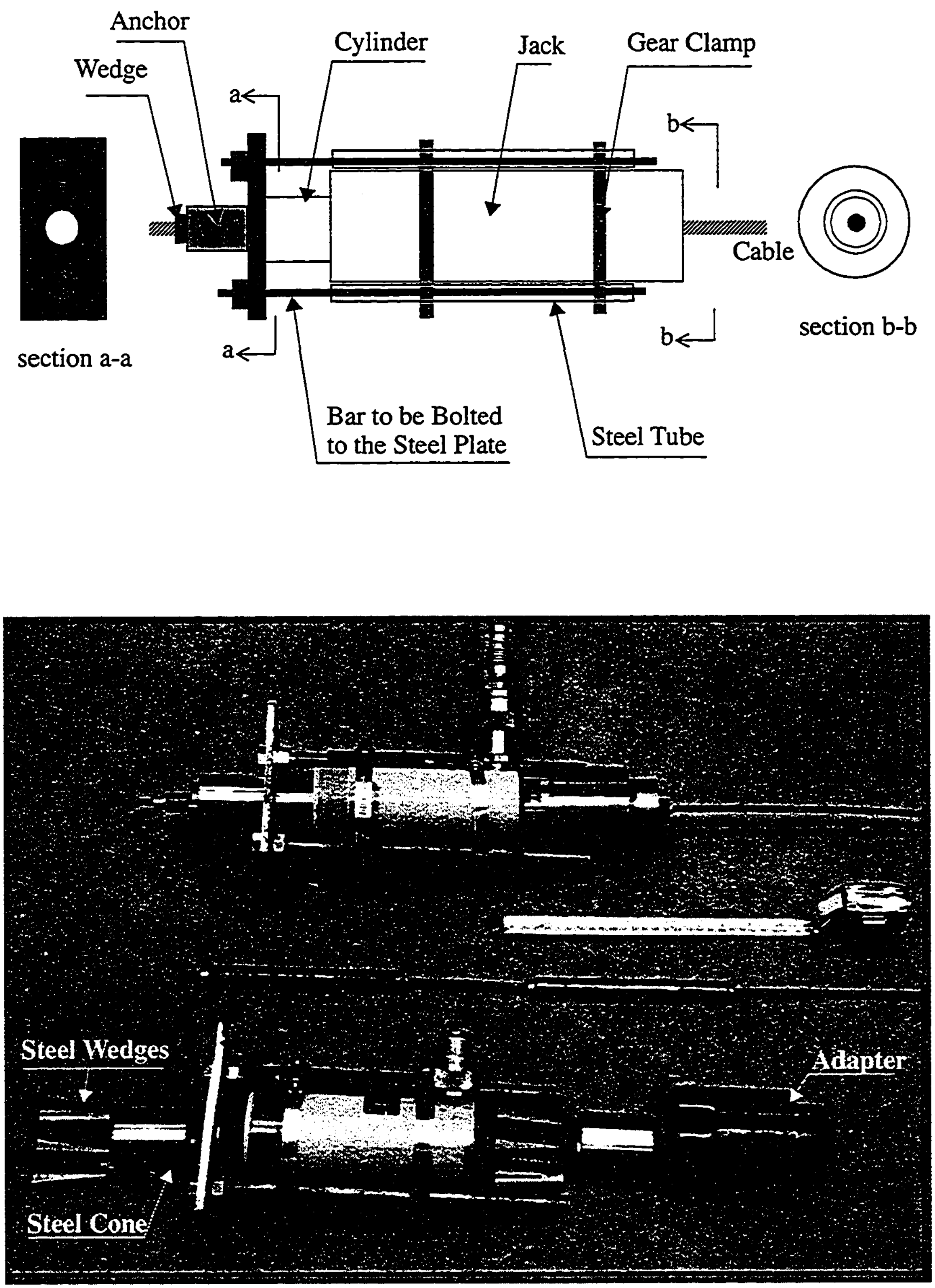


Figure 3.13 A Rotation-Stop Device Prepared to Prevent Unwinding of CFCC during Prestressing

3.6.2.4 Magnitude of the Jacking Force

The strength of CFCC very often decreases when the cable is tensioned while bent (or draped) as compared to the strength of the cable when tensioned while being straight. Therefore, care must be taken with regard to the magnitude of the initial prestressing force so that overloading will not take place. In case of straight cables, Tokyo Rope recommends to use an initial prestressing force equal to 0.6 times the guaranteed breaking load. The reduction in strength of a bent or draped CFCC depends on such factors as the radius of the bent, the bending angle and the portion of the cable in contact with the saddle. Although, Tokyo Rope does not provide guaranteed breaking load for draped prestressed CFCC, they recommend the use of a reduction coefficient α which depends on the aforementioned factors. For example, for a bending diameter-to-cable diameter ratio D/d = 48 and bending angle $2\theta = 10$ degrees, a reduction coefficient $\alpha = 0.8$ can be used. Thus, a jacking force of $0.6 \times \alpha \times P_u$, with P_u being the guaranteed breaking load of a straight cable, can be used.

3.6.2.5 Avoiding the Risks during Tensioning and Testing

Breakage of CFCC is sudden and occurs without warning by yielding. Therefore, the prestressing operation must be carefully carried out and the jacking force must be monitored with a load cell or oil pressure of the jack. Overloading must be avoided. Also, wedges may blow off accidently or due to breakage of CFCC. Therefore, protective shields or panels must be installed to prevent accidents.

3.7 Test Set-up

The test set-up for one of the twelve beam specimens is shown in Figure 3.14. During the test, each specimen was supported at one end on a roller support (Figure 3.15) and at the other end on a hinged support (Figure 3.16). All beams were subjected to two concentrated loads, each applied at one third of the span and increased monotonically in small increments. The total load was applied using a hydraulic jack and transmitted to the concrete specimen through a steel spreader beam. Two 2-inch diameter cylinder welded to the bottom of the spreader beam and spaced at 1/3 of the span of the concrete beam were used to produce the two concentrated load (Figure 3.17). A rubber pad was placed between the top surface of the concrete beam and each of the cylinders to aid in providing a uniform bearing surface. The two 2-inches cylinder were used for the first beam but later were replaced by a two pieces of monorail to increase the stroke of the jack as shown in Figure 3.14. The spreader beam was connected to a steel swivelhead attached to the hydraulic jack through a load cell.

3.8 Instrumentation

The beam specimens were instrumented to measure the applied load, deflection, concrete strains in the extreme compression and tension fibres and the strain in the tension reinforcement. Figure 3.18 shows a typical arrangement of deflection gauges and strain gauges for the beam specimens. The deflection and the strains were measured at the locations shown in the figure. Demec points were located in the midspan zone on each side of the beam centerline in order to measure the strain distribution over the depth of the beam Figure 3.19. The force in the external CFCC was also monitored during the tests by means of load cells.

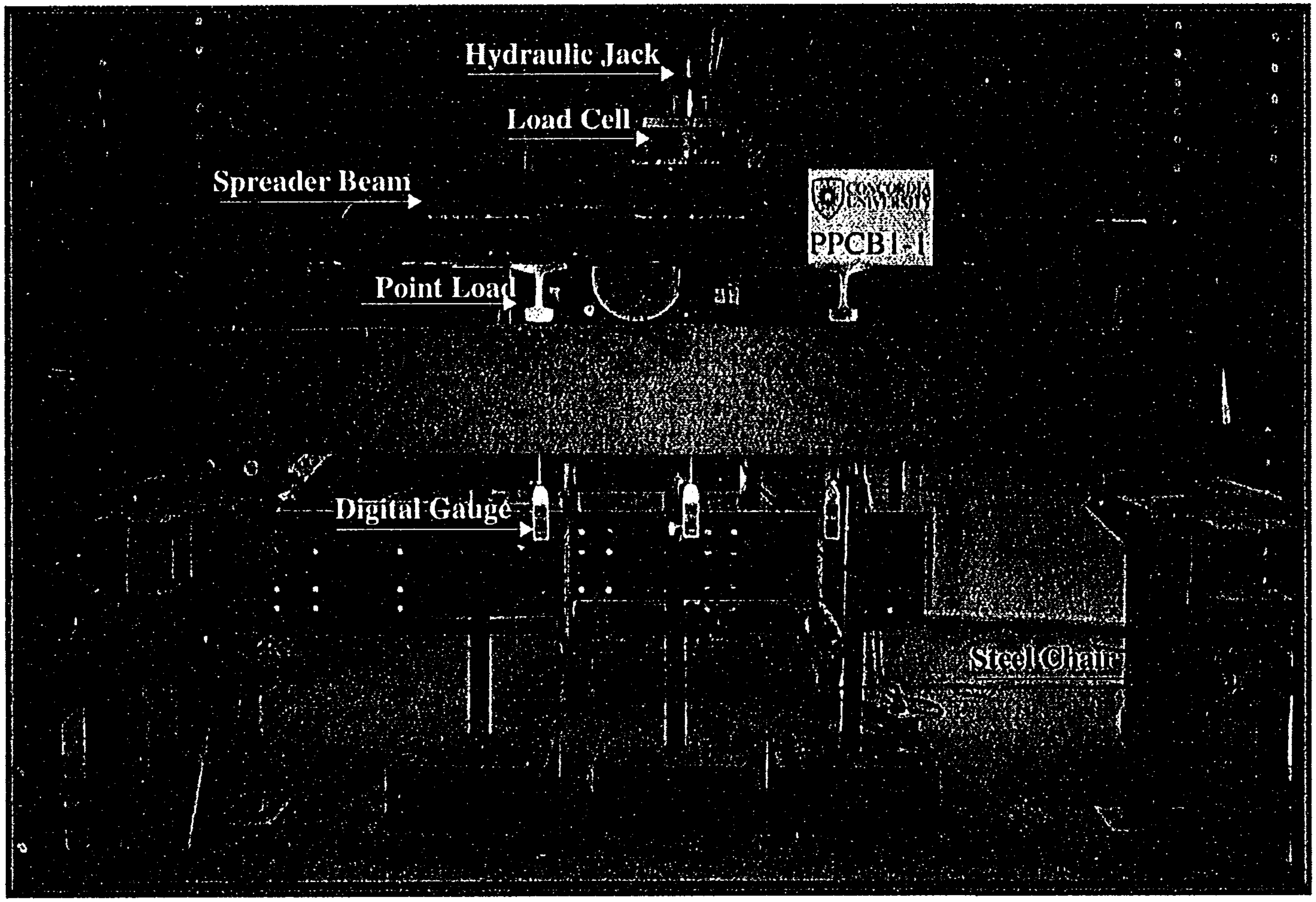


Figure 3.14 Test Set-up

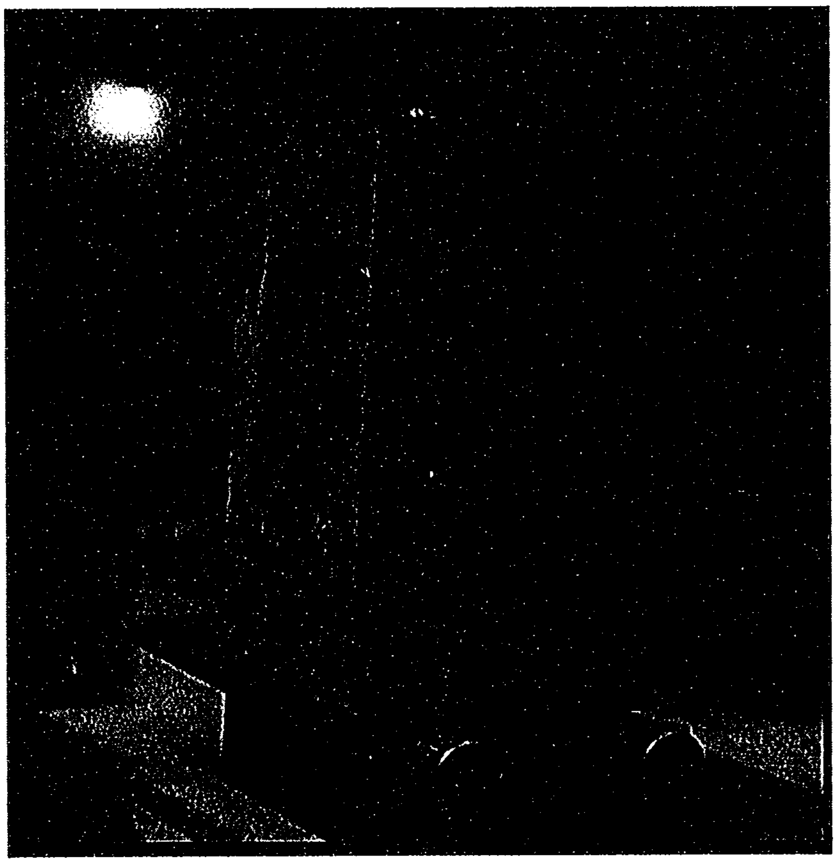


Figure 3.15 Roller Support

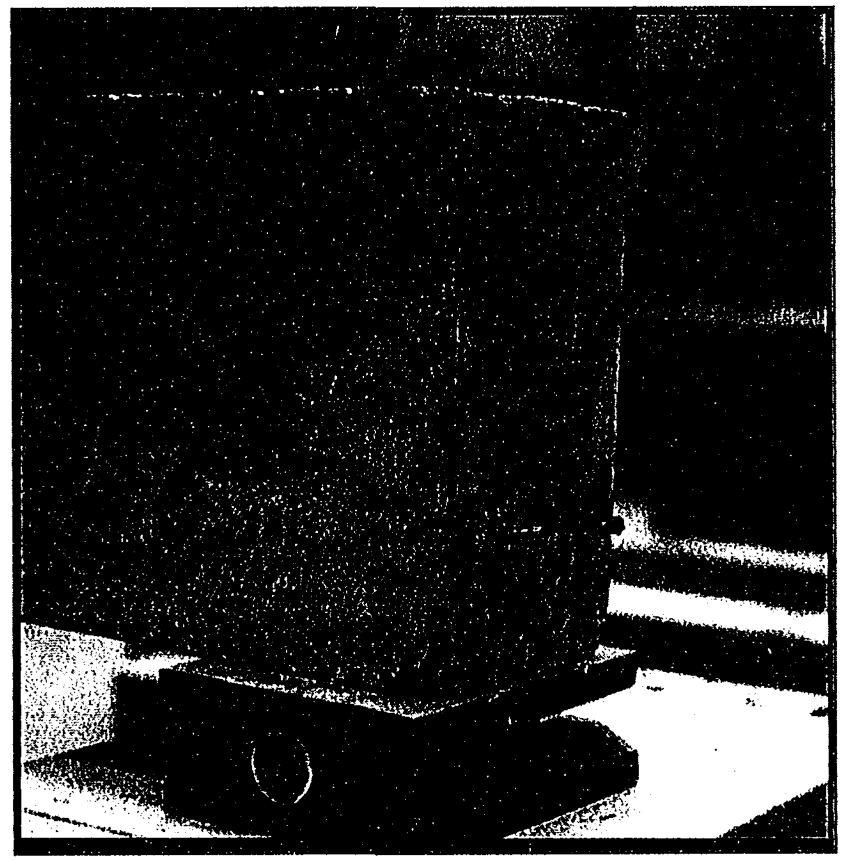
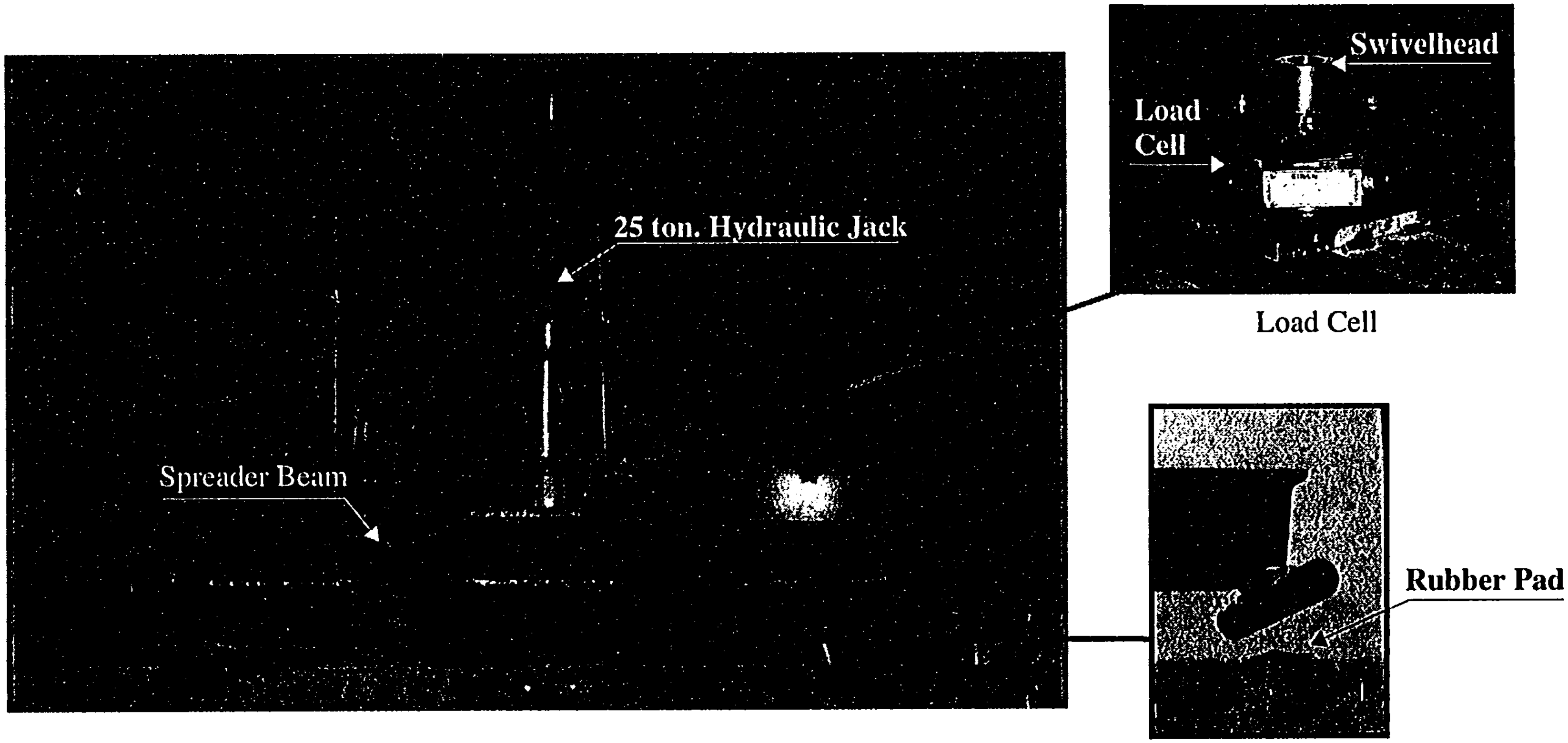


Figure 3.16 Hinged Support



Load Cell

2 in. Diameter Cylinder

Figure 3.17 Spreader Beam

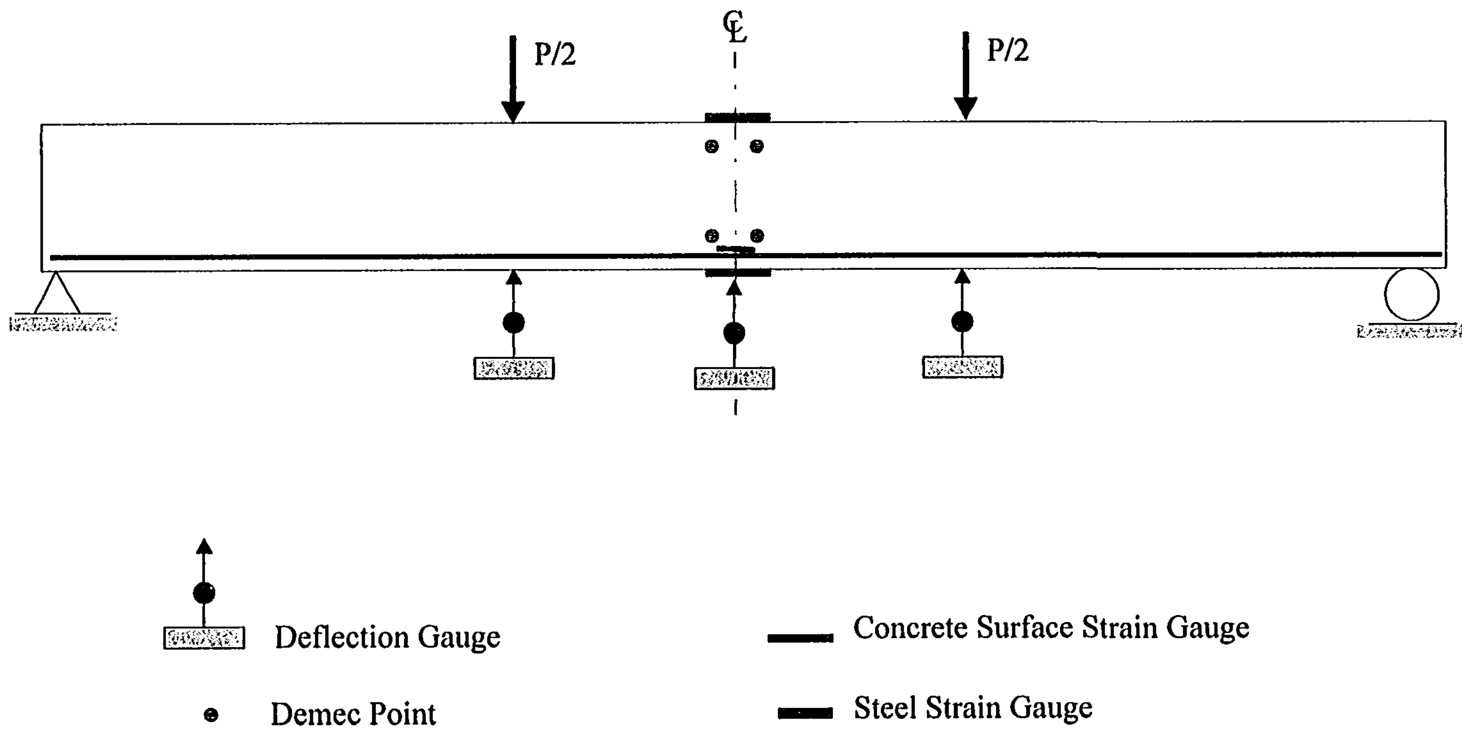


Figure 3.18 Instrumentation Plan

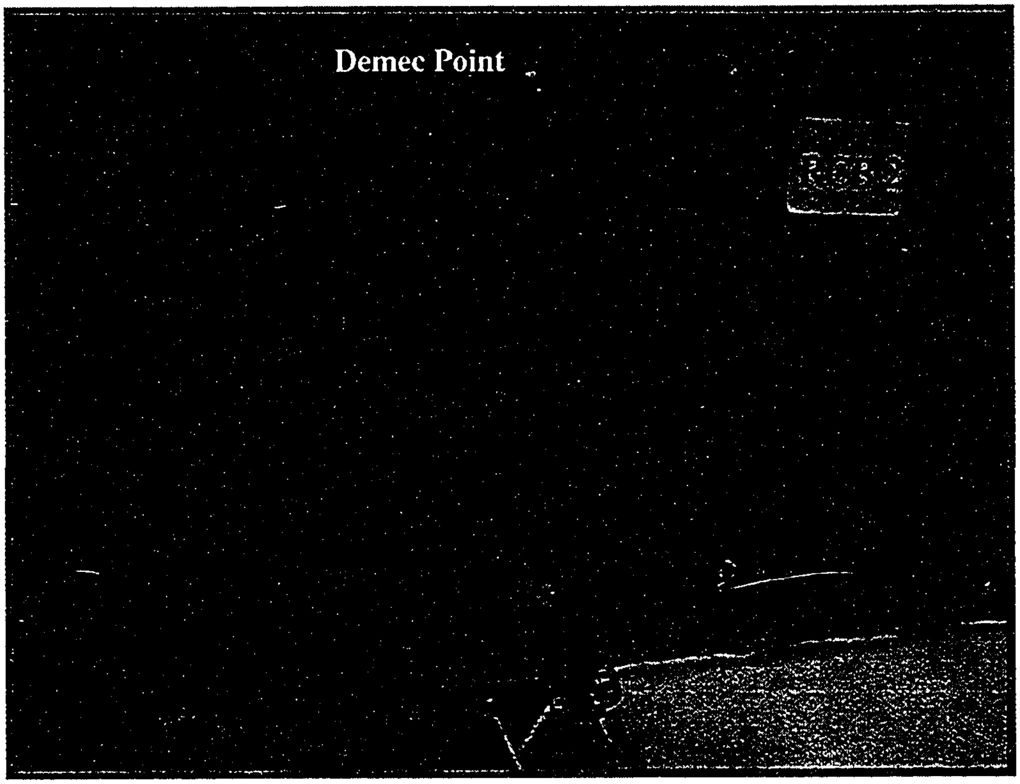


Figure 3.19 Demec Points

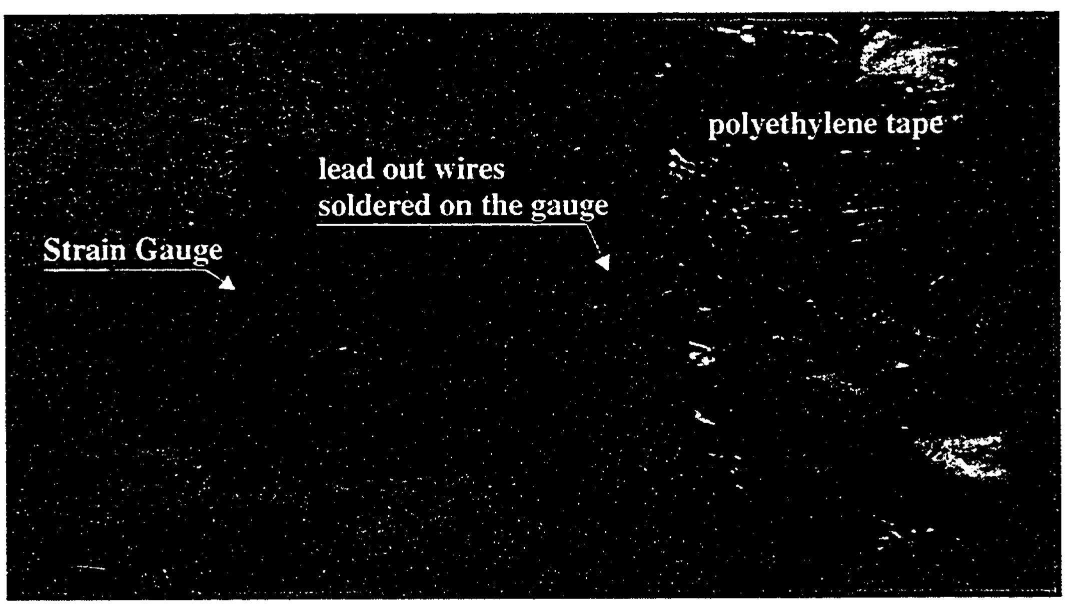


Figure 3.20 Concrete Strain Gauge

3.8.1 Strain Gauges

As mentioned earlier, that prior to casting the beams, electrical resistance strain gauges were attached to the flexural reinforcing bars at their midspan location. One strain gauge was used on each bar. The strain gauge used is of type N11-FA-60-120 of 10 mm long. The surface area of the reinforcing bars under the strain gauge was first cleaned of all paint and rust, and then smoothed with a fine grade emery paper or fine sand blasting to provide a sound bonding surface. The surface area was then degreased with solvant such as trichlorethylene and finally neutralised with a weak detergent solution. The method of strain gauge installation was as follows: after cleaning of the surface area, a small amount of adhesive was spread uniformly by brush or spatula on the surface, the gauge was pushed gently into position, and then covered with a piece of polyethylene sheet and a pressure was applied over the whole length of the gauge. Since the lead out wires from the gauge are fragile, care was taken to prevent any excessive forces from being applied on the location where the lead out wires are soldered on the gauge. The strain gauge was checked before applying a coating material to insulate the strain gauges. A rubber band was wrapped around the coat to ensure additional waterproofing and protection of the strain gauges against the soft concrete (Figure 3.3).

The concrete strain at the compression and tension faces were measured using two 120 Ω electrical resistance linear strain gauges of type EA-06-20CBW-120 of 60 mm length installed at midspan on the top and bottom faces of each beam (Figure 3.20). In order to monitor the beam centre tension and compression strains, a P-3500 portable digital strain indicator was used and readings were taken for each load increment.

3.8.2 Deflection Gauges

Digital displacement measuring devices with travel sensitivity of 0.01 mm were used to measure the deflection of the beams at three locations during application of the load. Three digital gauges were used for each beam; one gauge was placed at midspan, and one was placed directly under each of the loading points (Figure 3.14). Each gauges was supported during the test by a magnetic base attached to a vertical steel tube welded to a rigid steel channel. Figure 3.21 shows the set-up for deflection gauges.

3.9 Testing Procedure

Prior to the actual test, the beams were loaded initially to a small fraction load and then unloaded. The purpose of this step is to stabilize the beam and to prevent any possible twisting during the actual test. The readings of the strain gauges and the deflection gauges were then brought back to zero. The actual test was implemented in two stage. In the first stage, the beams were subjected to a load large enough to induce cracking and deformations close to the maximum allowable under serviceability limit state. The load was then removed and the beams were left for some time under their own weight in order to experience some time-dependent deformations. The purpose of this stage is to simulate the actual conditions of concrete flexural members that require strengthening and/or rehabilitation. In the second stage, the external prestressing using the CFCC was applied and the beams were subjected to a monotonically increasing load up to failure.

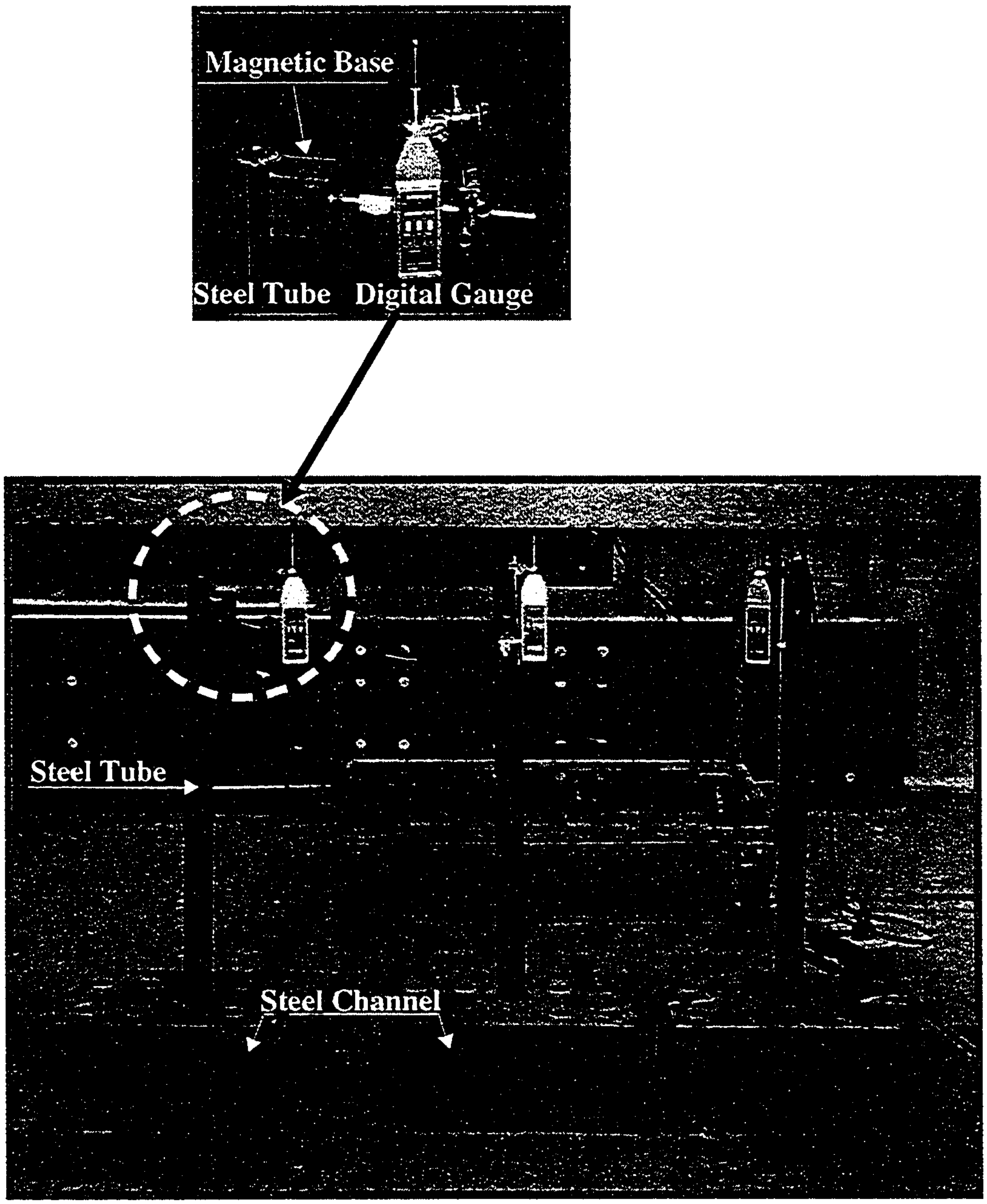


Figure 3.21 Digital Deflection Gauges Set-up

In the first stage, the total load applied on each beam was of such magnitude that produced deflection at midspan equal to 1/460 of the span length. The magnitude of the total load differed from one beam to another depending on the span length and the amount of internal prestressing. In both stages, the load was applied in small increments. All measurements such as beam deflections, the concrete strain at the top and bottom faces of the beam, strains in the reinforcing steel, the crack widths and spacings and the force in the external cables were recorded. The cracks were marked on both sides of the beams. Two different colors of markers were used to differentiate between cracks occurred in stage 1 before strengthening and cracks propagated or took place in stage 2 after strengthening. The time necessary to record the readings and mark the cracks after each load increments was three to five minutes. The duration of each test for stage 1 loading was from one hour to one and half hours. After the load of stage 1 was removed, some of the beams were kept under their own weight to monitor the effects of creep and shrinkage of concrete and the relaxation of prestressed steel and the deflections and cracks opening were recorded (Figure 3.22).

3.10 External Prestressing Procedure

Each beam specimen was externally prestressed using two Carbon Fiber Composite Cables, one on each side of the cross-section center line. Each cable was located at 25 mm away from the side of the cross-section (Figure 3.1). The two cables were stressed at the same time to ensure equal distribution of the total prestressing force to each cable. A single-point draped profile of the external prestressing cables was used. The depth to the bottom of the deviator at midspan was 324 mm and the eccentricity of the cables at the anchorages was zero. A typical view of the externally prestressed CFCC at the end of the external prestressing operation is shown in Figure 3.23.

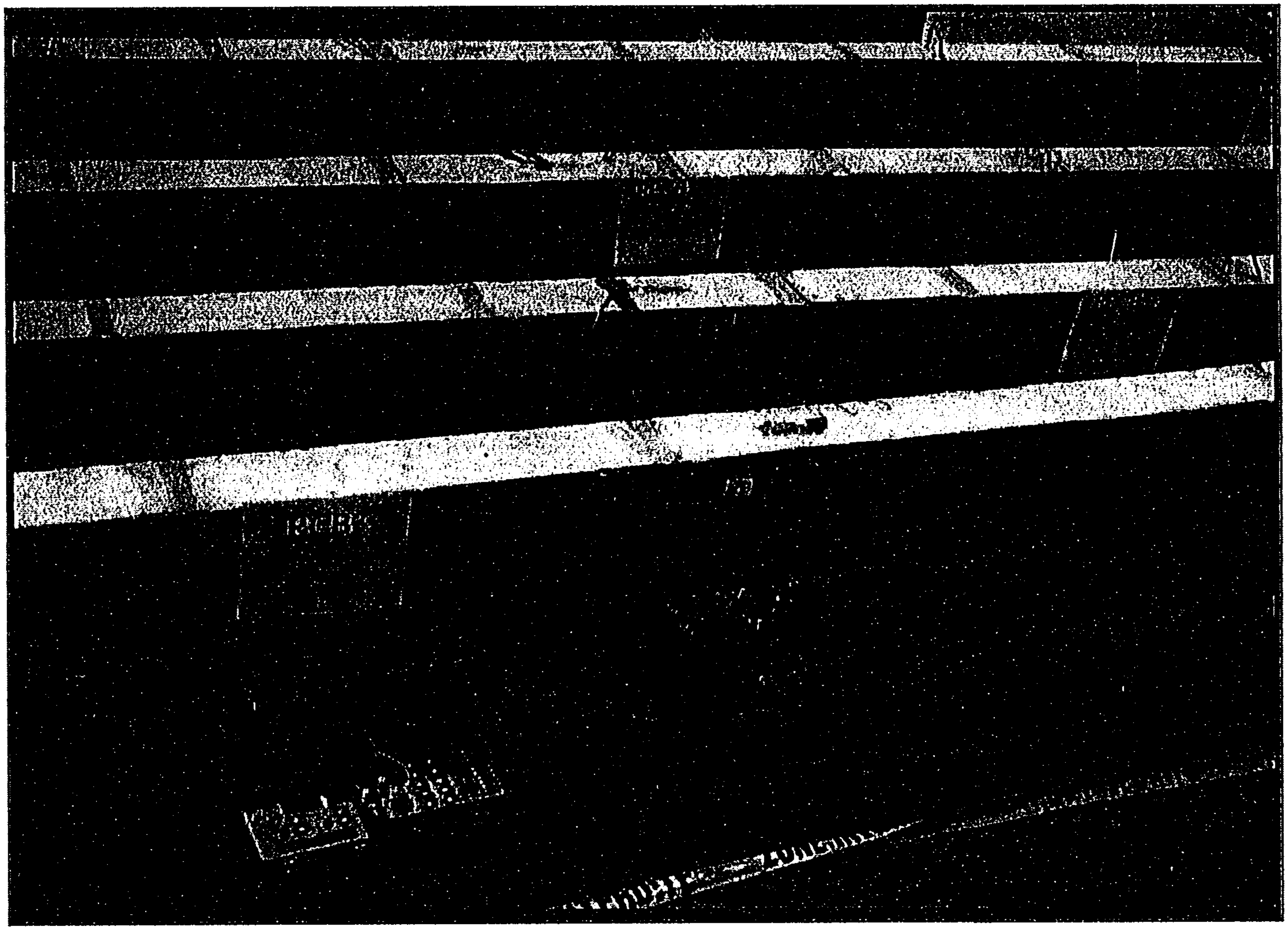


Figure 3.22 Measuring of Deflection During Test Under Time Dependent Effects

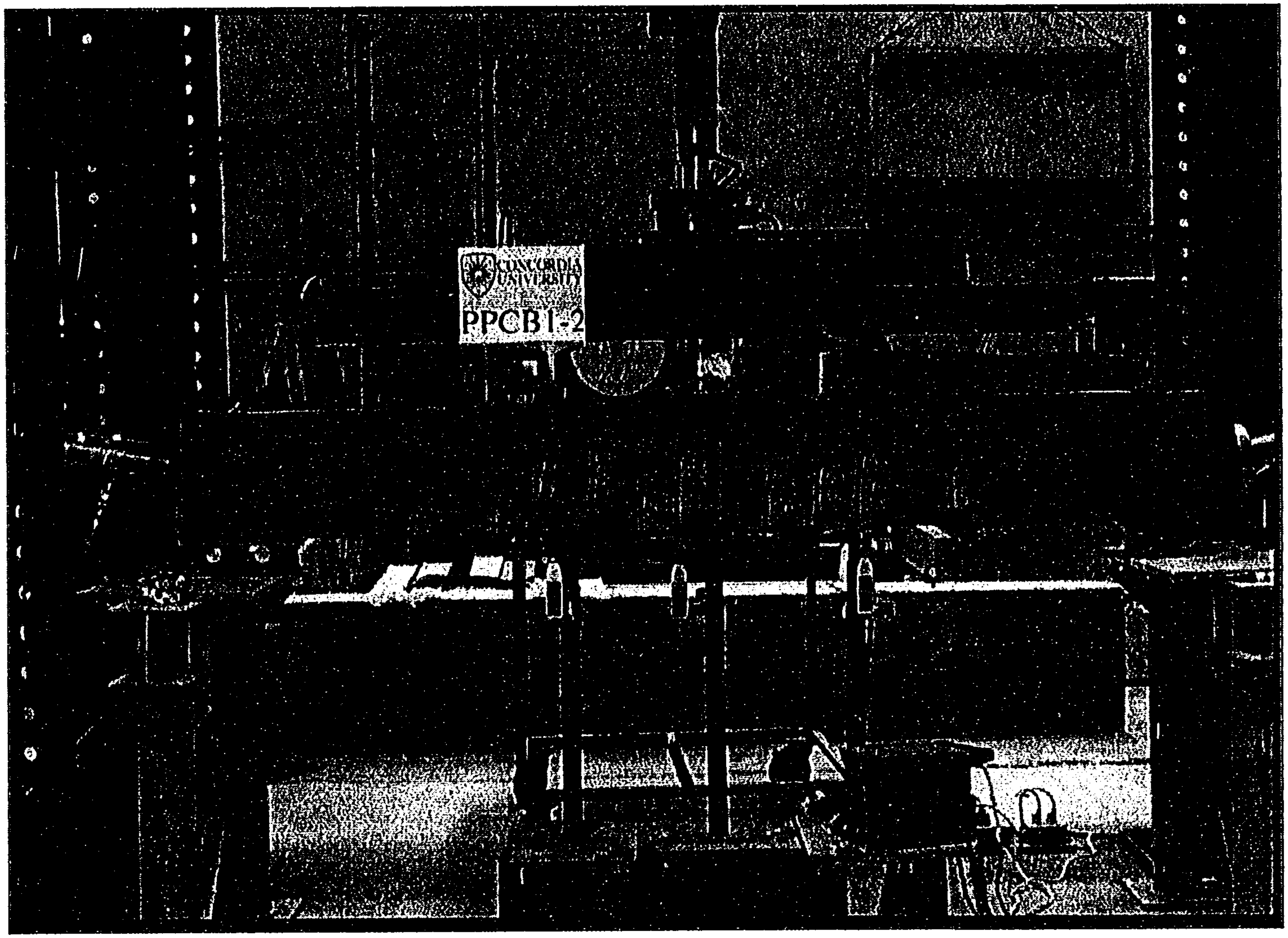


Figure 3.23 Typical View of External CFCC Prestressing Cables

Two identical jacks were used, one for each tendon, both connected to the same hydraulic cabinet. The prestressing force was introduced in the two cables simultaneously in small increments with the two hydraulic jacks from one end until the desired prestressing force was reached. The target effective prestressing force in the cables was about 50% of the guaranteed breaking load. The current practice is to keep the jacking stress within 60% of the nominal tensile strength of FRP prestressing reinforcement (JSCE 1993, CHBDC 1996). Special care was exercised to balance the prestressing force in the cables to avoid biaxial bending of the specimens. The prestress force applied to the cables was monitored closely using load cells placed at the dead end anchorages. Dial gauges were used to detect any slip of the cables at the end section. The cables were placed in position through 280×380 mm bearing plates attached to each end of the beam specimen. Because of the inclination of the cable, a tapered bearing plate was used. Details of the end bearing plate are shown in Figure 3.24.

The dead end of each cable contained one die-cast passing through the hole of the bearing plate and fixed using the anchorage device provided by Tokyo Rope. At the jacking end, the jack is held away from the beam by means of an adapter, which allows access to the terminal anchorage device placed inside to fixed the wedge on the die-cast of the tendon in position in its stressed stage when we reach the desirable prestressing force. Force is applied by the jack, which brings the outside fixed wedge on the die-cast just in an opposite direction to the beam. The outer anchorage device, the jack and the adapter can be removed. The principles of the stressing procedure are shown in Figures 3.25 and 3.26 respectively.

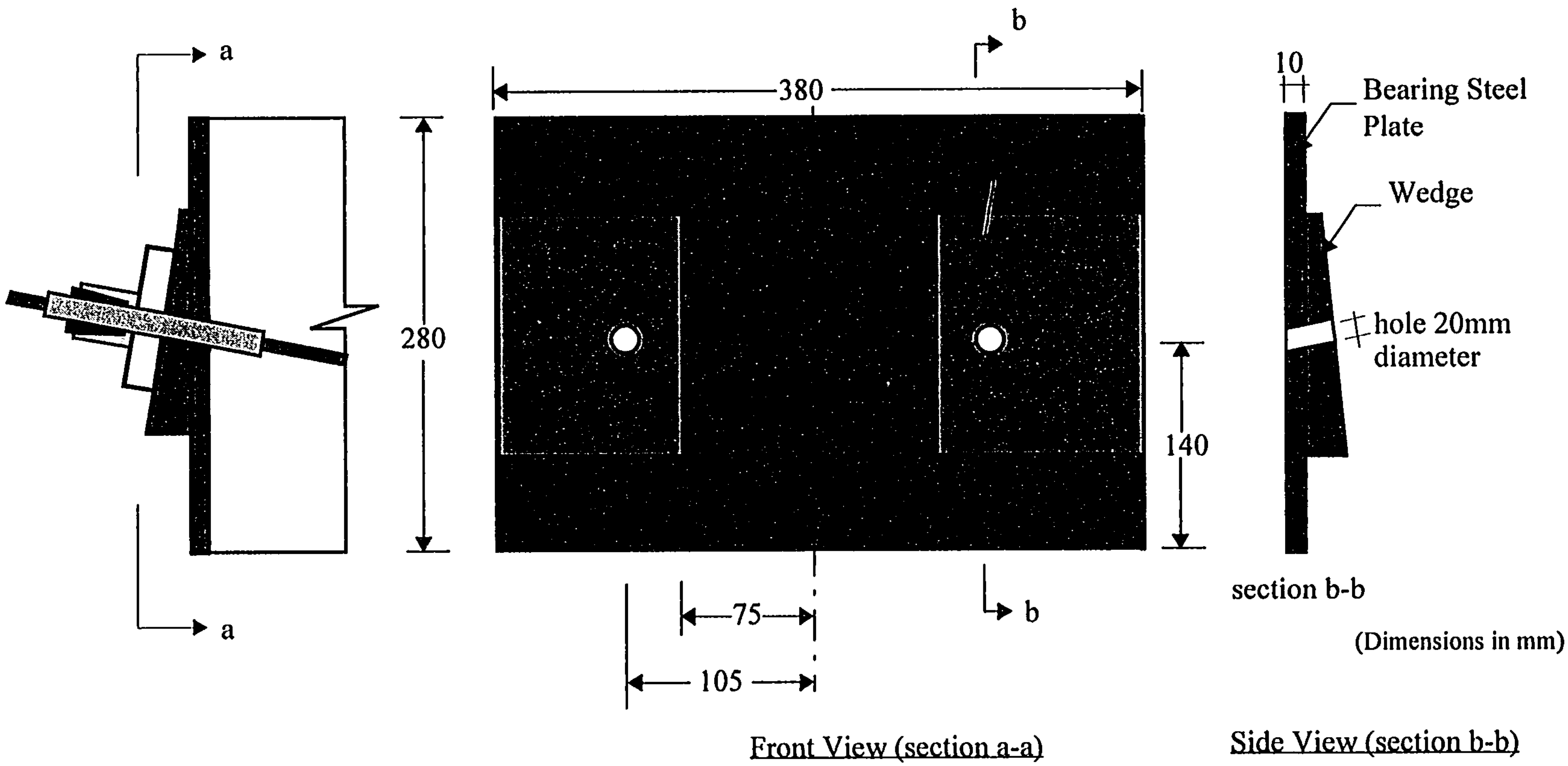
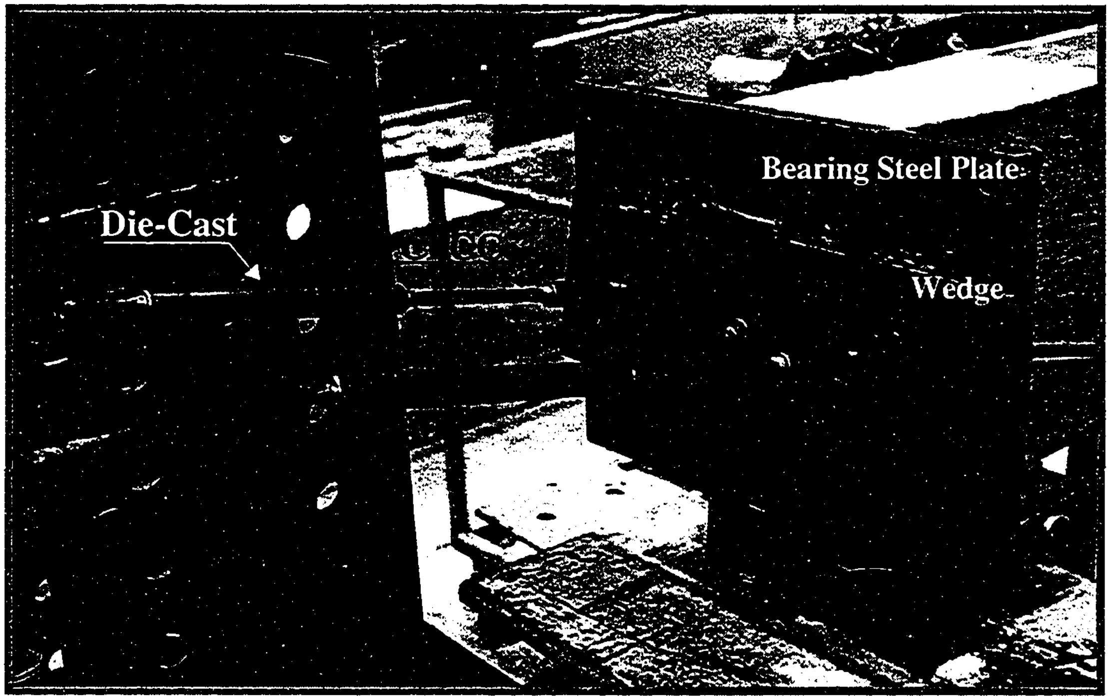
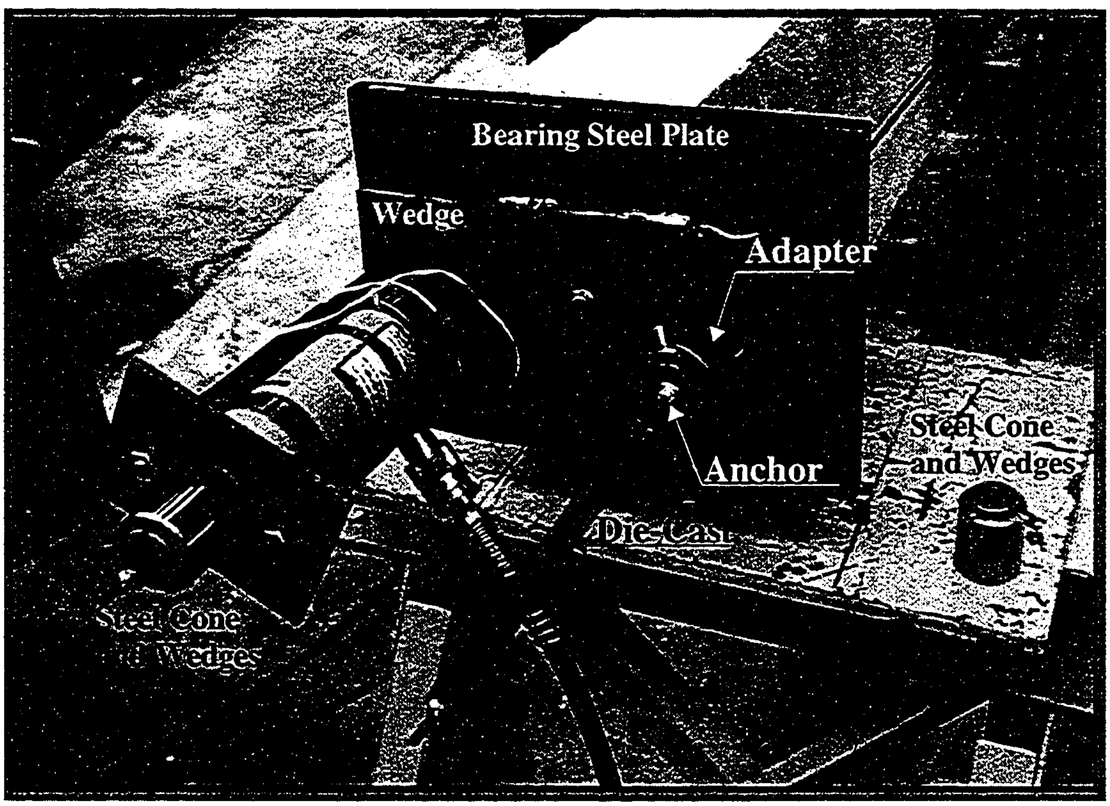


Figure 3.24 Details of the End Bearing Plate

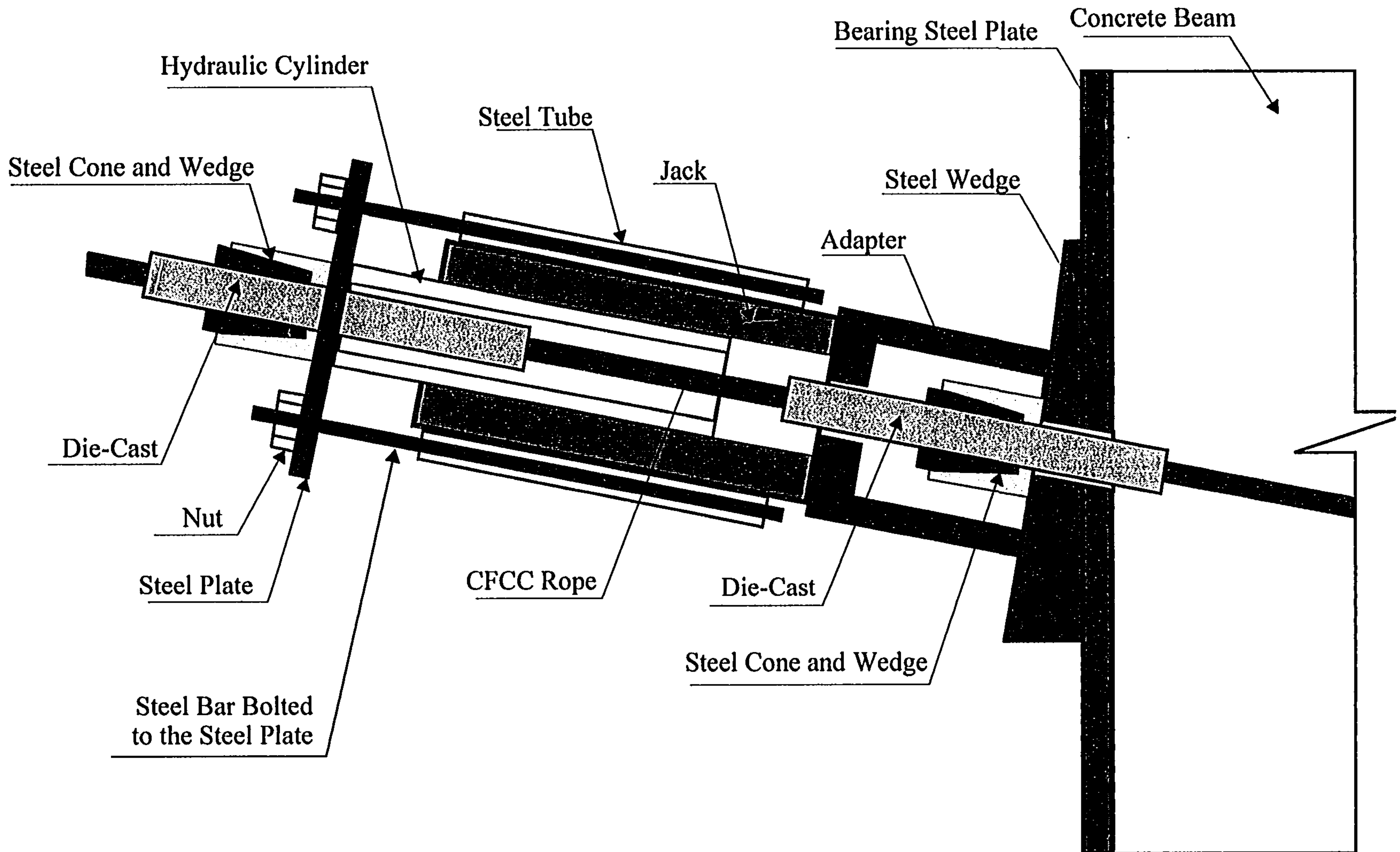


(a) Die-Cast at the Jacking End



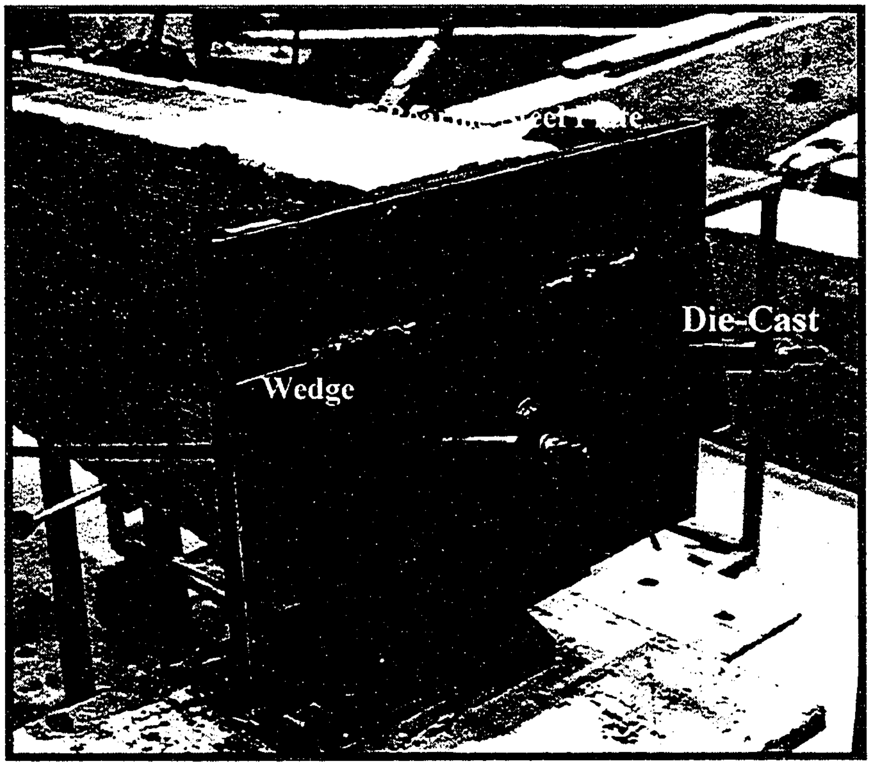
(b) Anchorage Set-up at the Jacking End

Figure 3.25 Prestressing Set-up at the Jacking End

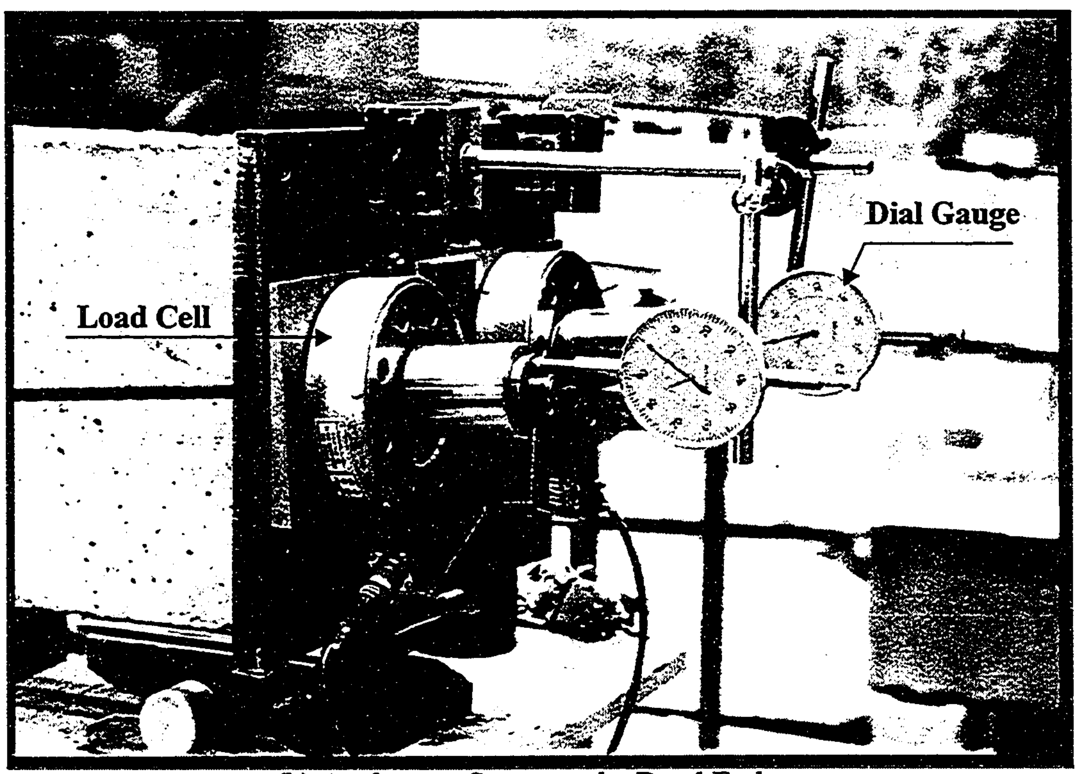


(c) Details of Anchorage Set-up at the Jacking End

Figure 3.25 Prestressing Set-up at the Jacking End (cont'd)

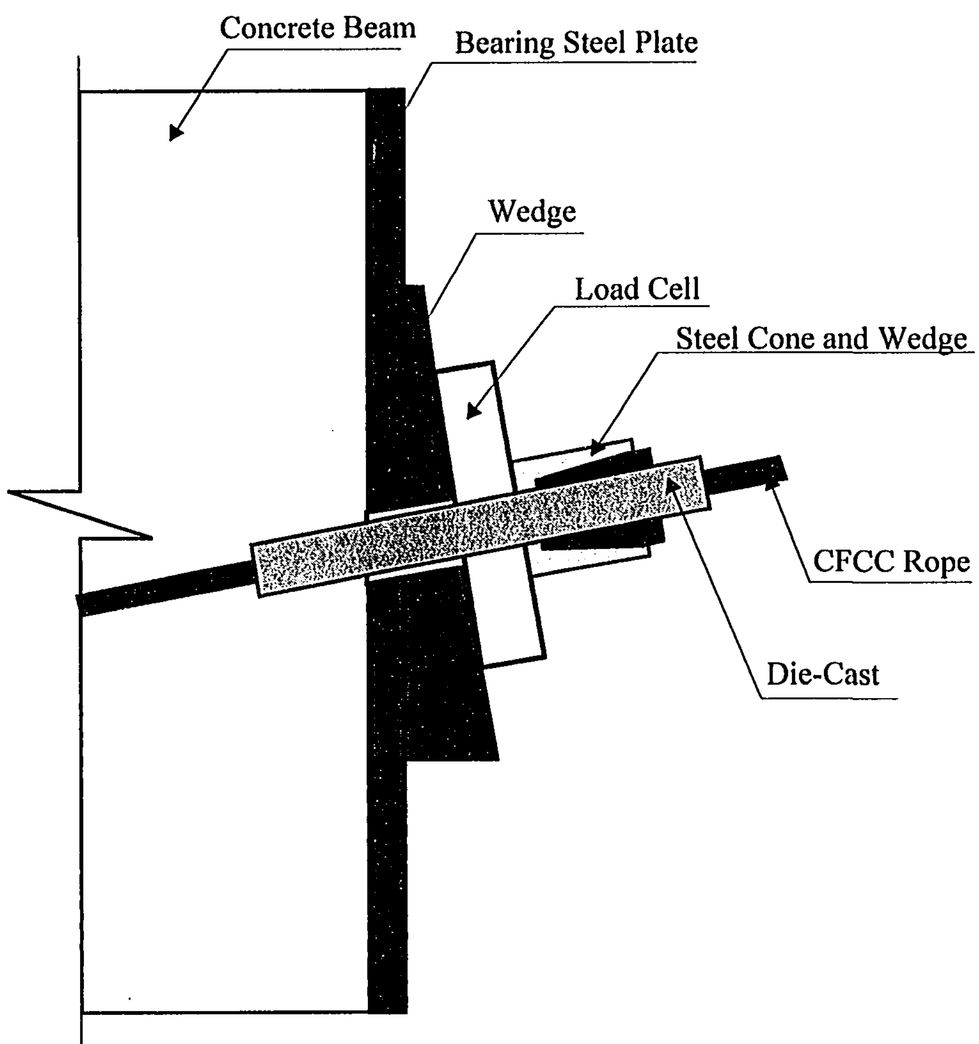


(a) Die-Cast at the Dead End of the Cable



(b) Anchorage Set-up at the Dead End

Figure 3.26 Prestressing Set-up at the Dead End



(c) Details Of Anchorage Set-up at the Dead End

Figure 3.26 Prestressing Set-up at the Dead End (cont'd)

Table 3.7 gives the experimental variables for the external CFCC cables. The tensile stress was given by the manufacture based on tensile strength test, as well as the values for the guaranteed breaking loads. These values are less than 60% of the nominal tensile strength of each cable as specified by the manufacture. They represent about 48% of the nominal tensile strength of the cable.

Table 3.7: Introduced Prestressing Force in External Cable of Beams in Group 1*

Beam Designation	Cross-sectional area: mm^2 (diameter: mm)	Nominal Tensile stress of cable (MPa)	Guaranteed breaking load (kN)	Introduced prestress force in cable (kN)
RCB1	10.1 (5.0)	2120	18	10
PPCB1-1				
PPCB1-2	30.4 (7.5)	2040	57	30

*same data for beams in Group 2 and 3

Chapter 4

Experimental Results

4.1 General

In Chapter 3, an experimental program to investigate the behaviour of partially prestressed concrete beams after being strengthened with externally prestressed carbon fiber composite cables (CFCC) was described. Twelve beam specimens including three control beams were tested. Different partial prestressing ratios and span-to-depth ratios were considered. In this chapter, the results of the experimental program are presented and discussed. Plots of the load-deflection response of all the beam specimens during loading before and after strengthening with the external CFCC cables are presented. Graphs of the variation of the strain in concrete, the strain or stress in the reinforcing and the change in stress in the external CFCC cables with the applied load in all beams are also presented. The ultimate flexural strength and the modes of failure after strengthening are also discussed and compared with those of the control beams without strengthening.

Analytical equations are given for prediction the change in stress in the external CFCC cables at any load level and at ultimate. The values predicted by these equations are compared with those obtained from the tests. The effects of the span-to-depth ratio and the partial prestressing ratio on the behaviour of the tested beams, particularly on their load carrying capacity after strengthening and the change in stress in the external CFCC cables are discussed.

Full details of the experimental results over the entire history of loading of all beams are given in Appendix A. Photographs showing the beams before strengthening and at failure after strengthening are also given in the same appendix.

4.2 Load-Deflection Response

The behaviour of each strengthened beam was characterized by its load versus deflection response from zero up to failure. The specimens before and after strengthening were tested in four-point bending over a simply supported span. The two point loads were positioned at one third of the span. The beam vertical deflections at midspan and 1/3 of the span (under the point loads) were measured by means of digital gauges. As mentioned in Chapter 3, the load was applied gradually in increments in two stages:

Stage I: before strengthening, the load was applied to produce damage by cracking and permanent deformations. The load was increased until the deflection at midspan reached 1/460 of the span length and then decreased down to zero.

Stage II: The beams were reloaded from zero up to maximum load reached in stage I, then strengthened by external prestressing and then loaded further until failure.

Figures 4.1 to 4.9 show the load-deflection behaviour of all the twelve tested beams during stage I and II of loading. The curves shown in these figures are similar to the load-deflection response of flexural concrete members observed in many experimental studies. The graphs indicate that in the loading stage I, when the load was applied to produce cracking before strengthening, the behaviour was almost linear up to the initiation of

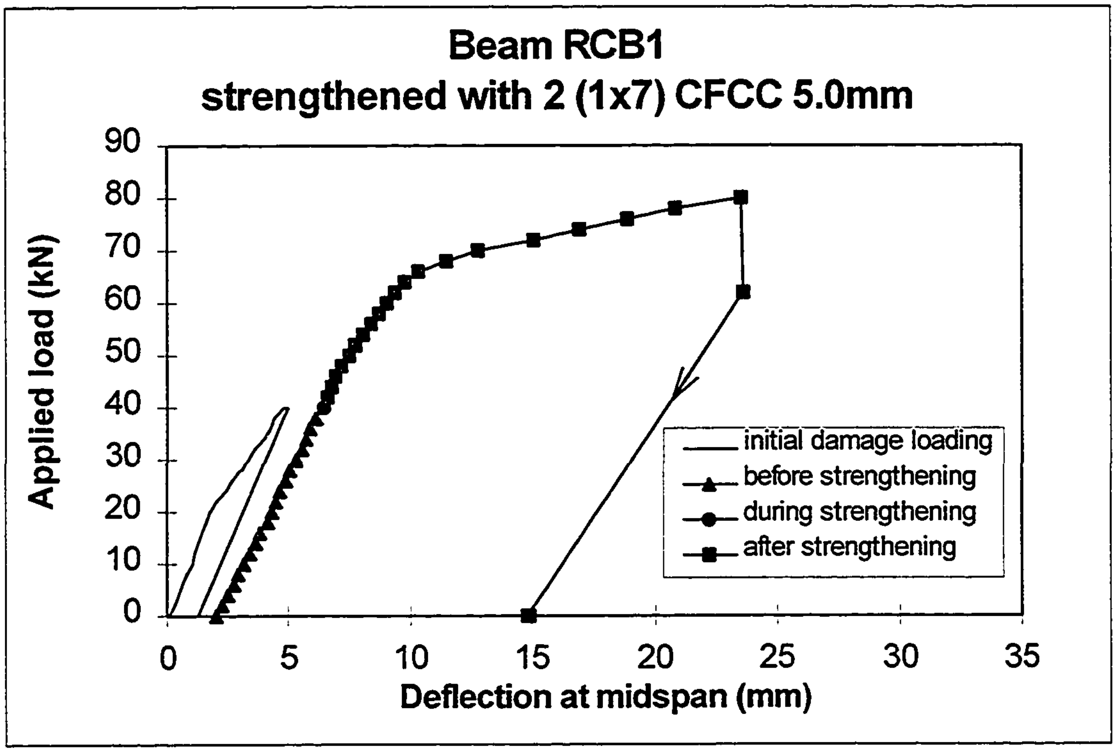


Figure 4.1 Load versus Deflection for Beam RCB1

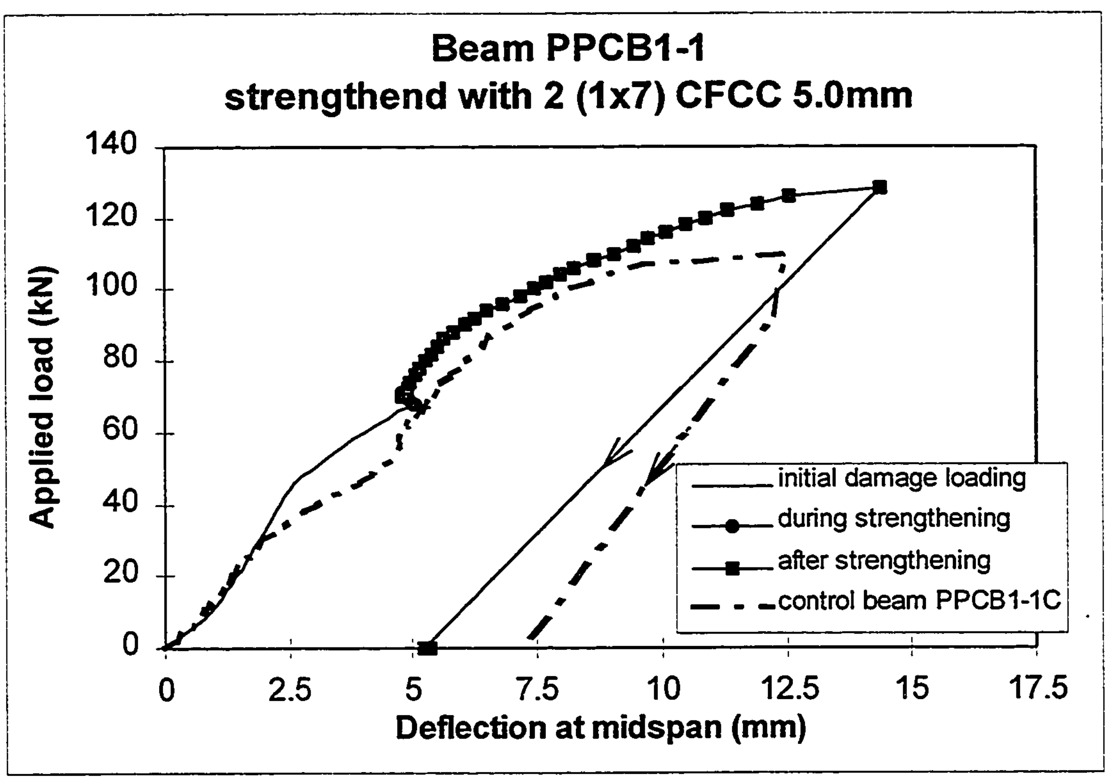


Figure 4.2 Load versus Deflection for Beam PPCB1-1

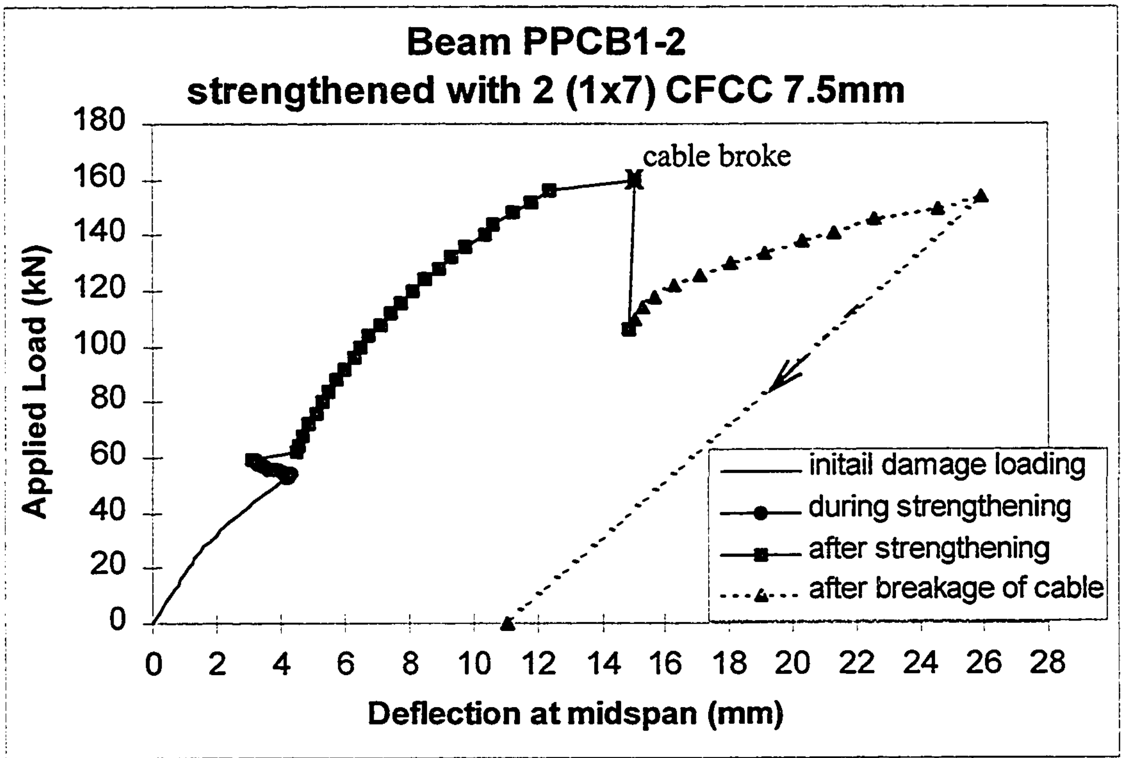


Figure 4.3 Load versus Deflection for Beam PPCB1-2

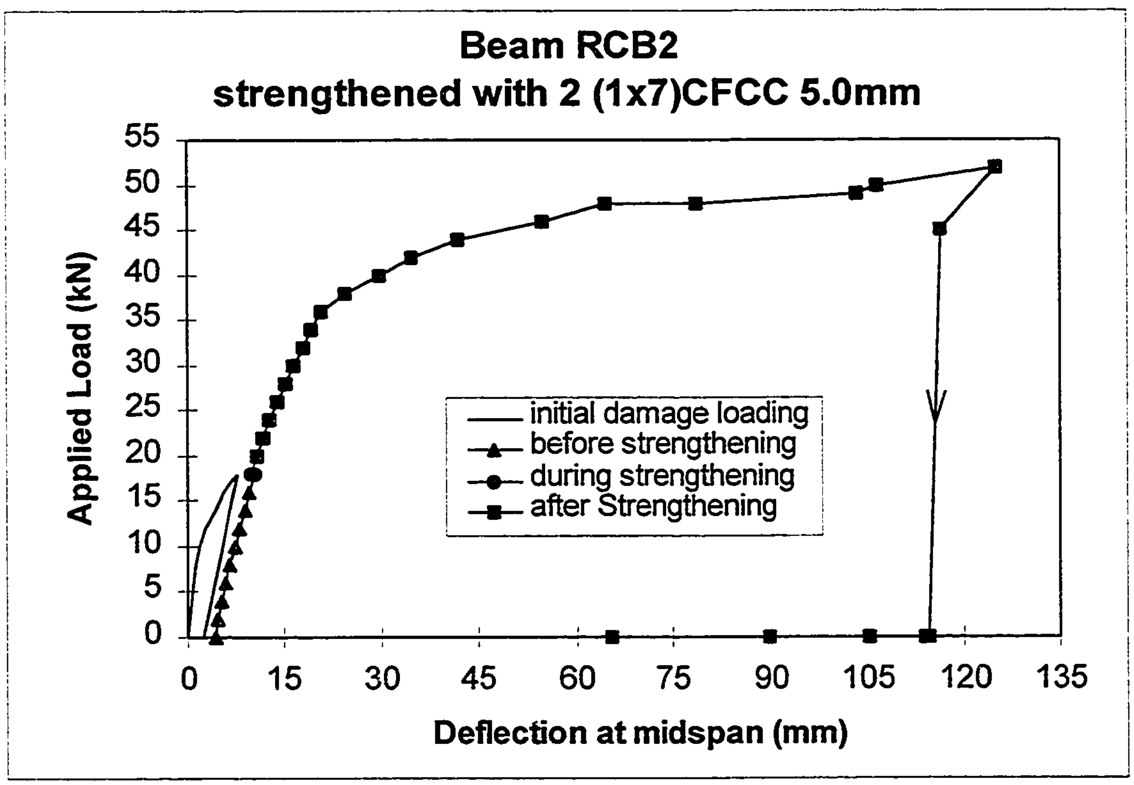


Figure 4.4 Load versus Deflection for Beam RCB2

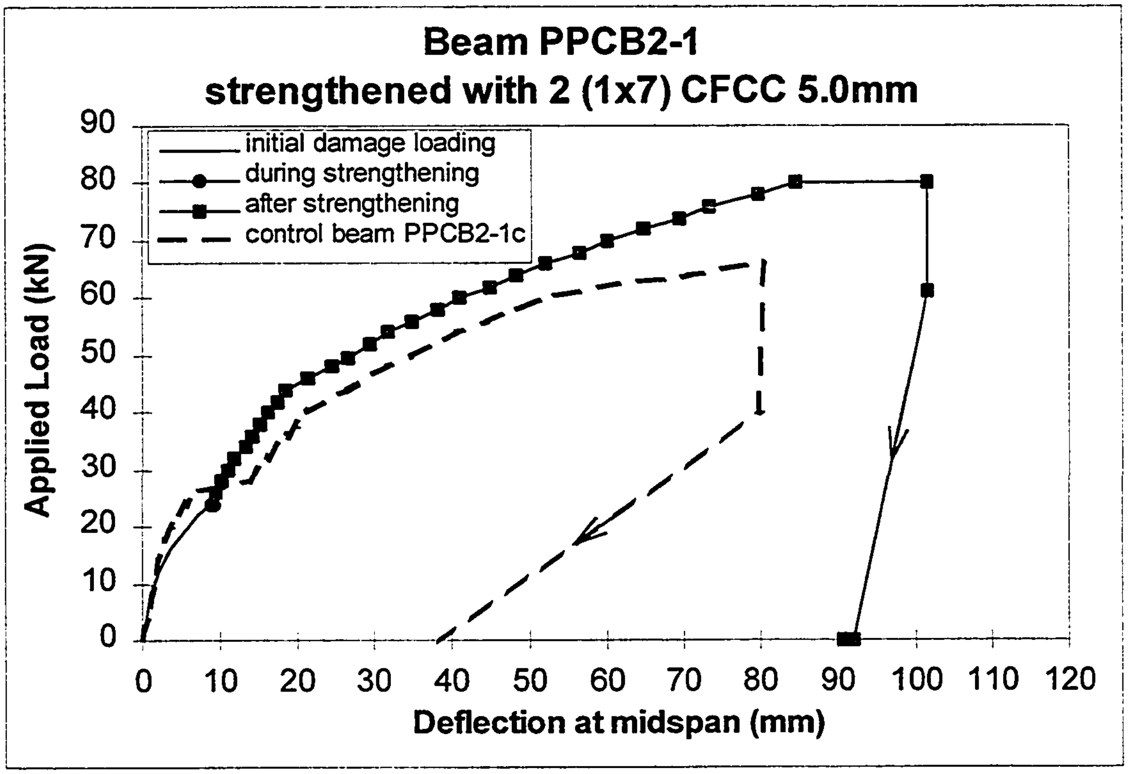


Figure 4.5 Load versus Deflection for Beam PPCB2-1

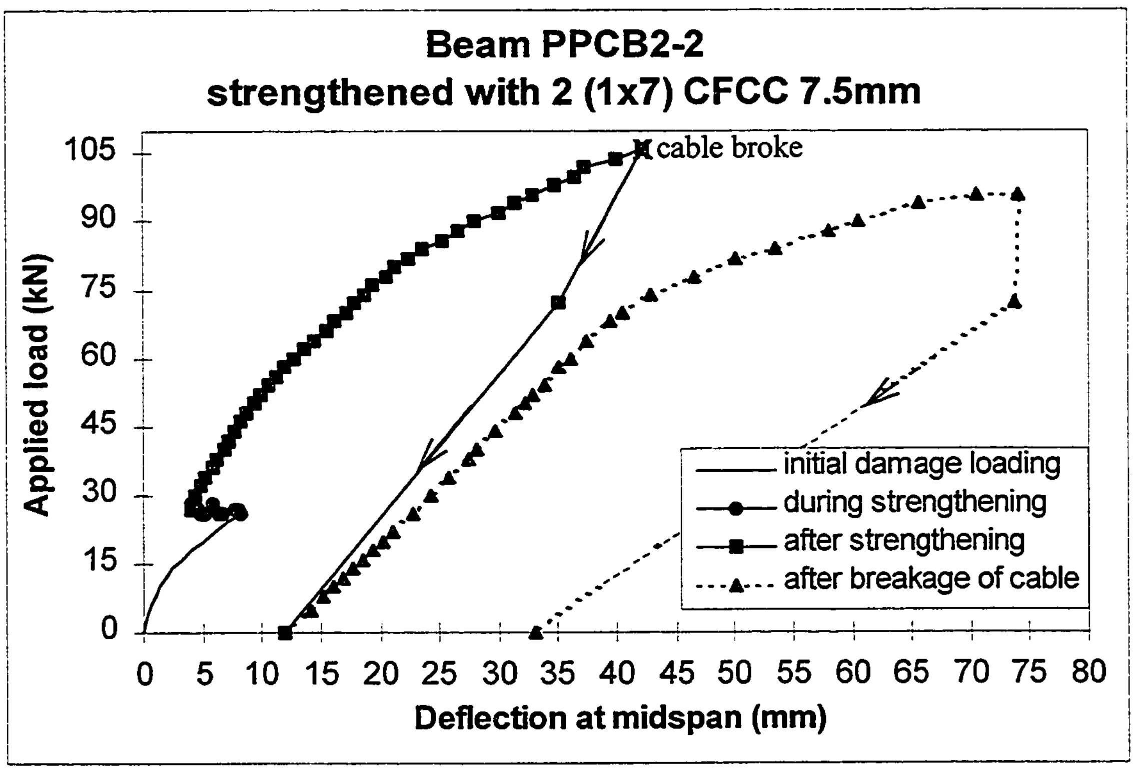


Figure 4.6 Load versus Deflection for Beam PPCB2-2

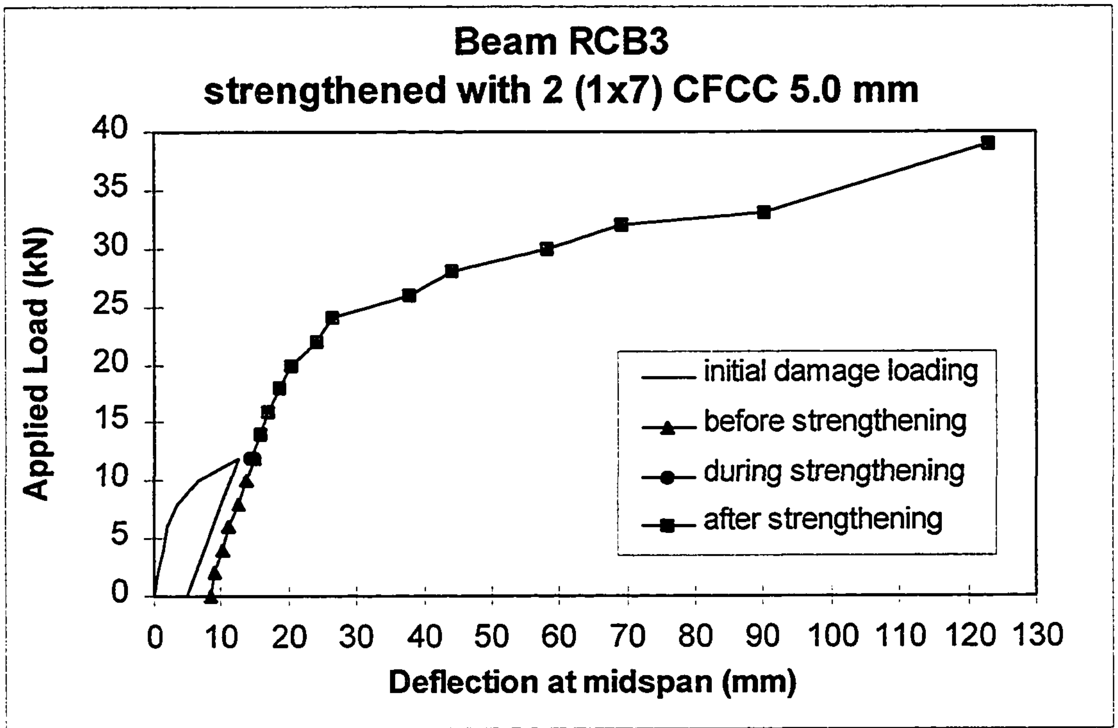


Figure 4.7 Load versus Deflection for Beam RCB3

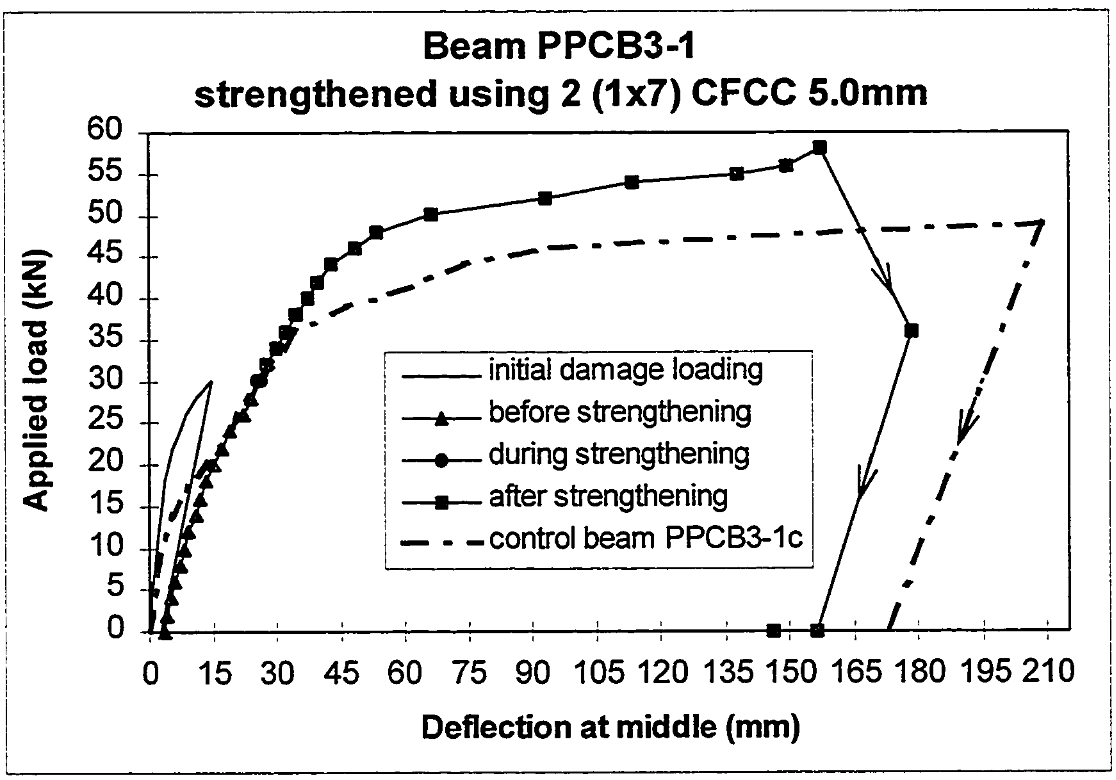


Figure 4.8 Load versus Deflection for Beam PPCB3-1

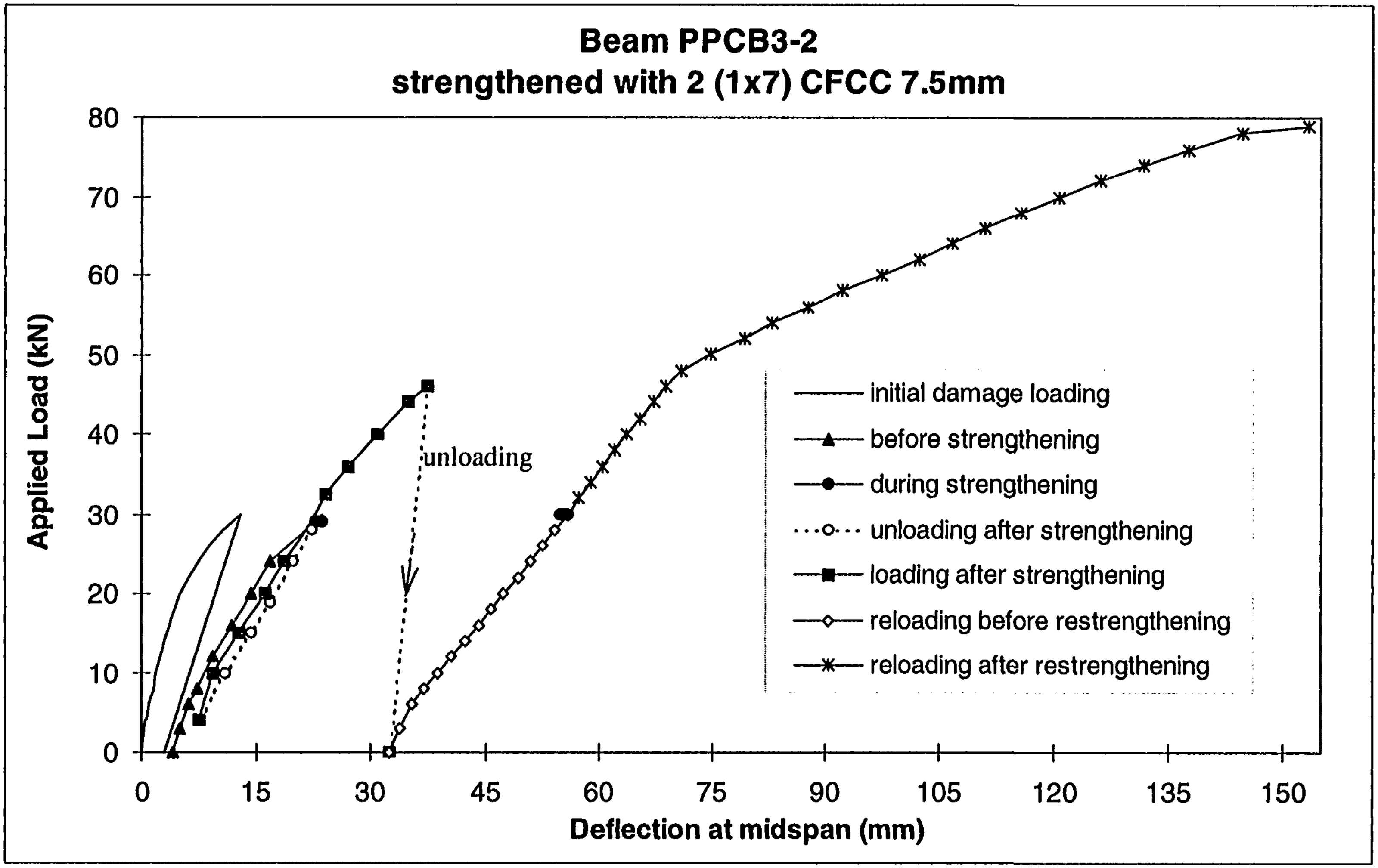


Figure 4.9 Load versus Deflection for Beam PPCB3-2

cracking. The flexural stiffness of the beam in this region is denoted as K_1 and depends on the amount of reinforcement in the cross-section and the span of the beam. After cracking, the stiffness is reduced to K_1'. Unloading in this stage down to zero resulted in permanent deformations at zero load. The increase in deflection at zero loading after unloading of stage I and just before reloading of stage II is due to the time-dependent effects of creep and shrinkage of concrete in the reinforced concrete beams in addition to relaxation of internal prestressed steel in the prestressed concrete beams. The beams in this case are subjected only to their own weight.

In stage II, before and after strengthening by external CFC cables, the load-deflection curves exhibited three zones of behaviour, one before external prestressing and two after application of the prestressing. These zones are described as follows:

1. During reloading and before application of the external prestressing, the flexural stiffness K_2 of the beams was slightly smaller than the stiffness K_1 before cracking in stage I, but greater than the stiffness K_1' after cracking of stage I.
2. After application of external prestressing, the cracks started to reopen and propagate under increasing the load and the flexural stiffness K_3 became smaller in comparison to the stiffness K_1 of the uncracked members.
3. Under further increase in the load, the cracks became wider and new cracks started to initiate until the load reached a value at which the tension reinforcement started to yield. The flexural stiffness decreased further until the ultimate load was reached at which failure took place either by crushing of the concrete in compression or by breakage of the external CFC cable.

During the application of the external prestressing, some of the beams experienced a decrease in the downward deflection (i.e., camber) at almost the same maximum value of the applied vertical loads reached at the end of stage I.

In general, the cracking load and the flexural stiffness expressed as the slope of the load-deflection response after cracking and the behaviour in the post-elastic range for the different beam specimens tested depend mainly on the partial prestressing ratio and the reinforcing index. It was observed that beams with higher *PPR* had higher cracking loads. This can be attributed to the larger effect of the prestressing force. Also, because of the larger amount of the tension reinforcement provided in the partially prestressed beams compared to the minimum value required by the code, the flexural stiffness of the beams increased with increasing the reinforcing index.

The following observations and conclusions were made from the experimental study:

- All beam specimens strengthened using external prestressing CFCC cable failed in flexural mode except for the beam with small span-to-depth ratio which failed in shear.
- In comparison with the control beams tested without strengthening (Figure 4.2, 4.5 and 4.8) the load-deflection response of the strengthened beams was improved. This was indicated by the increase in the stiffness of the concrete members after strengthening and reloading. At any load level, the deflection of the strengthened beams was smaller than the deflection of the beams tested without strengthening. It was observed that the external prestressing reduced the crack widths or closed the cracks completely, and led to a stiffer load-deflection response.
- The external prestressing improved significantly the ultimate flexural resistance of the concrete beams. An increase in the ultimate flexural resistance up 70% was obtained depending on the amount of internal prestressing and the span-to-depth ratio.

4.3 Modes of Failure

As mentioned in Chapter 3, all beam specimens were under-reinforced, and during the loading history, all beams showed sign of yielding before failure. Under increasing the applied load, the cracks became wider and new cracks started to initiate until the load reached a value at which the tension reinforcement started to yield. The flexural stiffness decreased further until the ultimate load was reached at which failure took place by crushing of the concrete in compression. The failure started when an increase in the deflection was observed without a significant increase in loading until collapse occurred in the beam. It was observed during the test, that the crushing of the concrete in the compression zone at failure was followed by a sudden and significant drop in the load resistance of the beam specimens. All the failure modes are shown by photograph in Appendix A.

In general, the failure of beams group 1 (S/d_p = 10.7) was in shear, and a secondary slip crack developed near the support. Beams Group 2 (S/d_p = 17.8), and Group 3 (S/d_p = 25) failed in flexural mode with a longitudinal crack developing at the top close to the point load directed towards the constant moment region. The flexural failure was initiated by the formation of a large flexural crack directed to the location of applied load. Almost symmetric cracking pattern was observed over the entire span.

Special cases for certain beams were observed as discussed in the following. After failure occurred in beams RCB2, RCB3, PPCB3-1, and PPCB3-2, an upward load from bottom was applied to help in releasing the external prestressing cables.

In beam PPCB1-2 ($S/d_p = 10.7$), the cable broke before failure occurred in the beams. At the instant of cable breakage the load dropped down. The load was then increased to cause failure in the beam itself. The beam exhibited a nonlinear behaviour up to failure and failed in shear.

The behaviour of beam PPCB2-2 ($S/d_p = 17.8$) was the same as beam PPCB1-2 but after breakage of the cable and the load dropped down, the applied load was removed totally, and the beam was retested later after 67 days up to failure. The beam exhibited a linear behaviour after reloading up to approximately 45% of the ultimate load, then exhibited a nonlinear behaviour up to failure and failed also in shear.

Beam PPCB3-2 ($S/d_p = 25$) was tested in a way different from the other beams. After the initial damaging load was applied and the beam was left for a period of time to experience some time-dependent effects under its own weight, the beam was loaded after strengthening up to 75% of the ultimate load of the beam without strengthening. The load was removed before failure occurred in the beam and then was reapplied after restrengthening until failure of the beam. The beam failed in flexure by crushing under the point load and a longitudinal crack in the compression zone at the top directed inside the constant moment region.

4.4 Concrete Strain

The Concrete strain was recorded at midspan over a gauge length of 60 mm using surface mounted strain gauges. Figure 4.10 to 4.18 depict the load versus the concrete strains at the top and bottom fibers of the cross section at midspan of the beams before and after strengthening with the external prestressing CFCC cables. The figure indicate that the load-top strain curves are similar to the load-deflection curves. The curves for load-bottom strain for beams PPCB1-1, PPCB1-2, PPCB3-1 and PPCB3-2 are not shown because of failure of the strain gauge during testing. As can be seen in the figures, during reloading after strengthening, the beams exhibited higher strains as compared to the first case of loading before strengthening. This continued up until the load at which a stabilized cracking pattern was reached. After this point, the strain increased at a faster rate with a reduced slope of the load-strain curve due to propagation of the cracks and reduction in the depth of compression zone of concrete and stiffness of the beams. The figures also indicate that the majority of the beams did not fail in compression (i.e., by crushing of concrete), meaning that the compression strain at the top fiber did not reached the ultimate compression strain of concrete before yielding of the steel reinforcement.

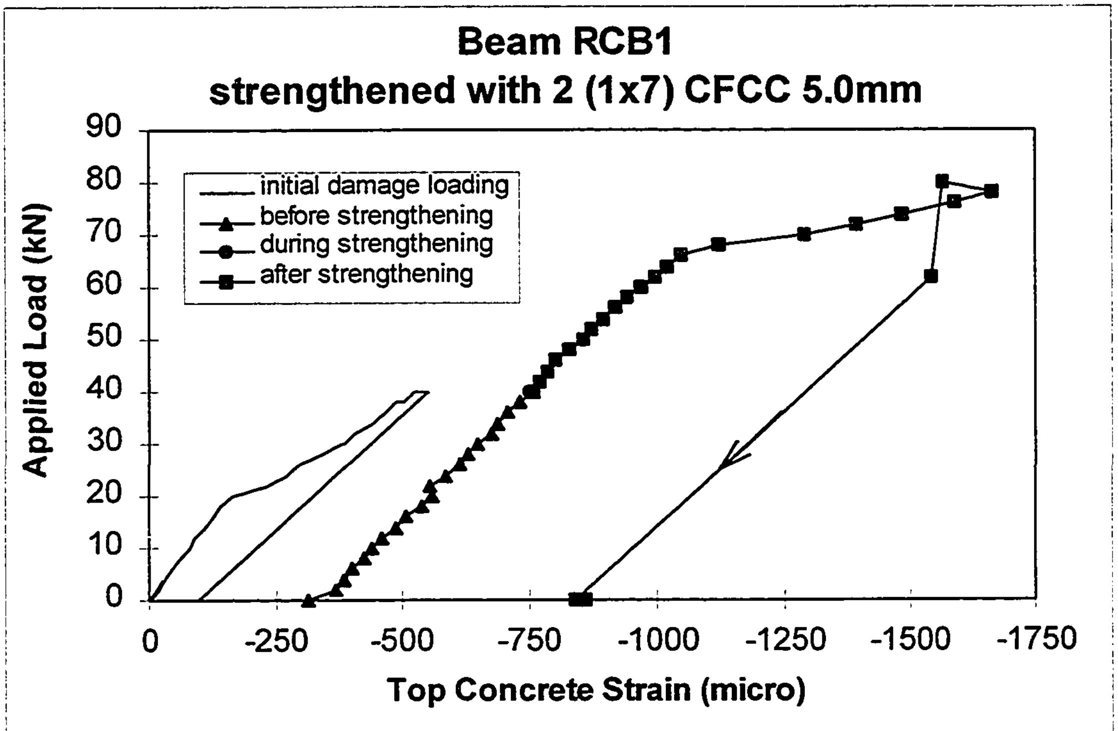


Figure 4.10a Load versus Top Concrete Strain for Beam RCB1

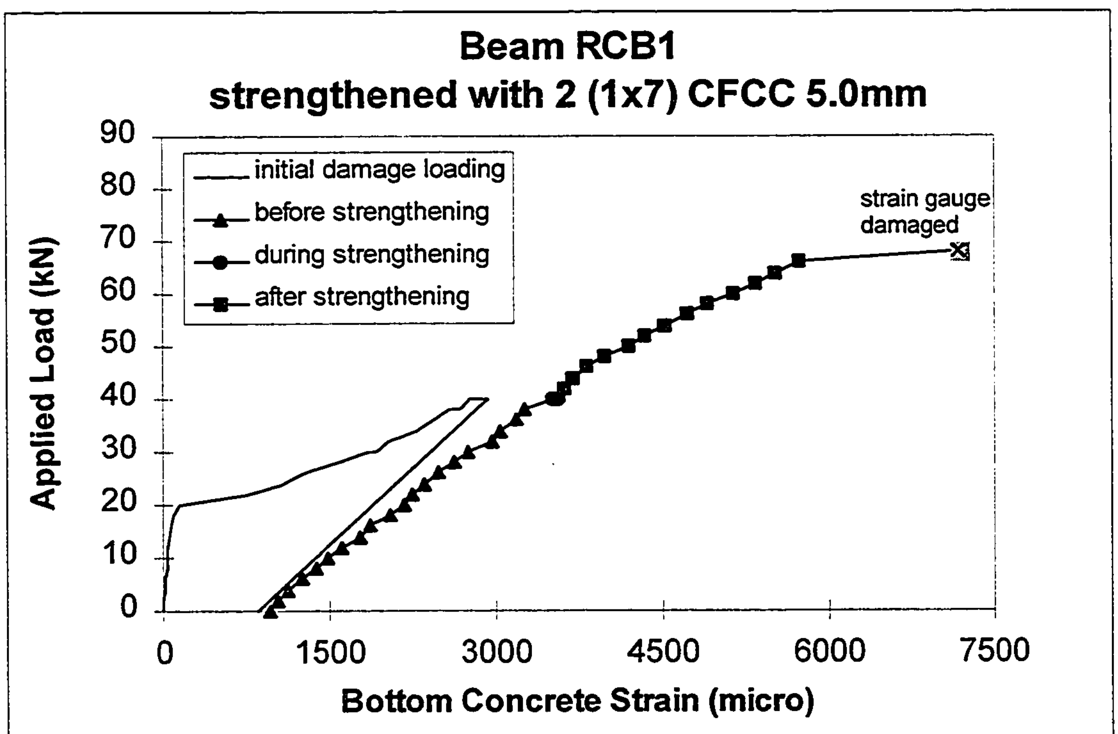


Figure 4.10b Load versus Bottom Concrete Strain for Beam RCB1

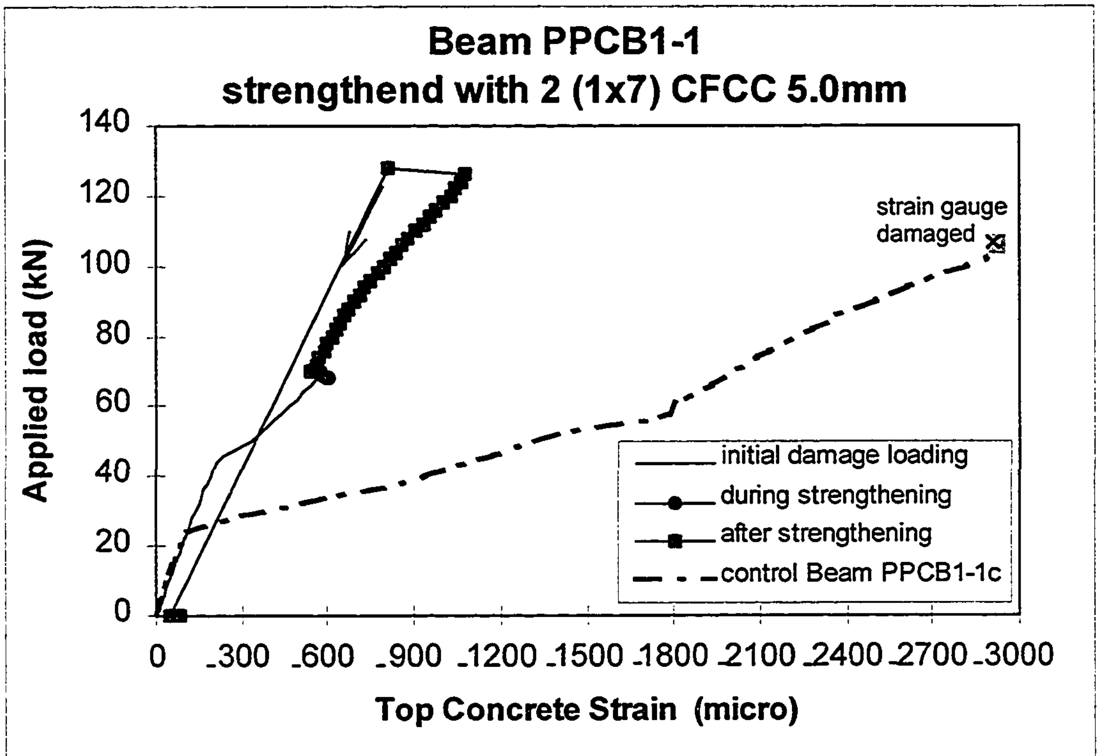


Figure 4.11 Load versus Top Concrete Strain for Beam PPCB1-1

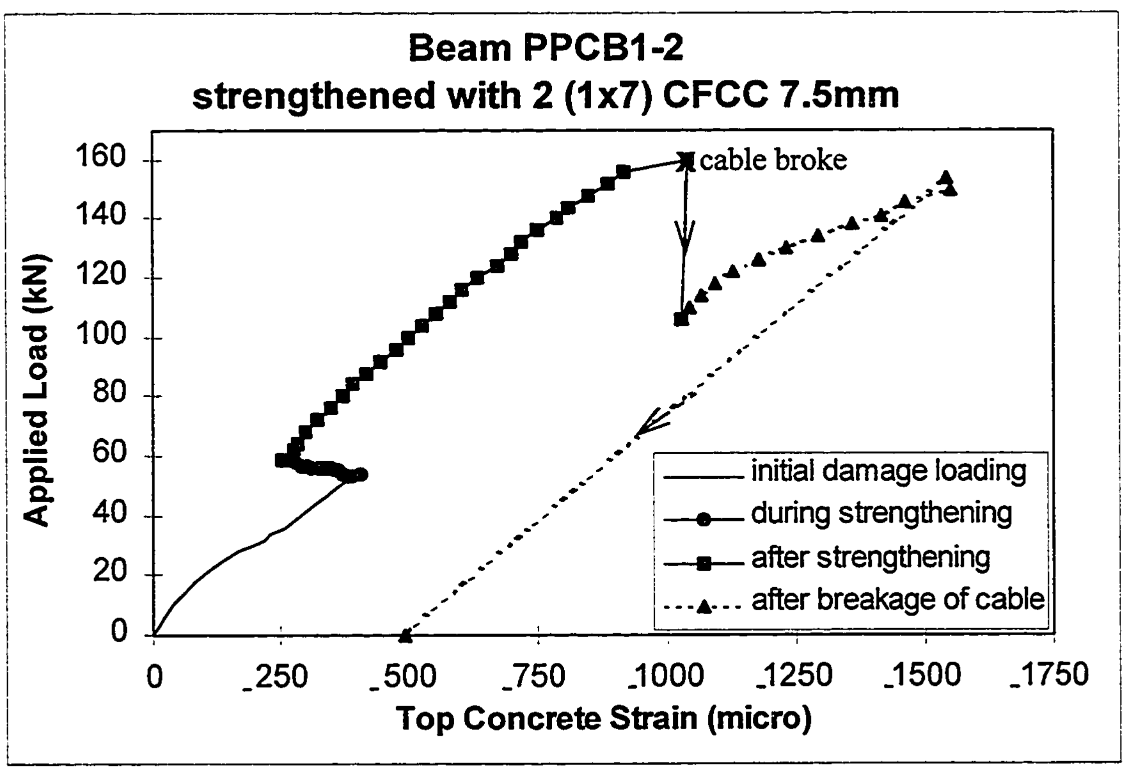


Figure 4.12 Load versus Top Concrete Strain for Beam PPCB1-2

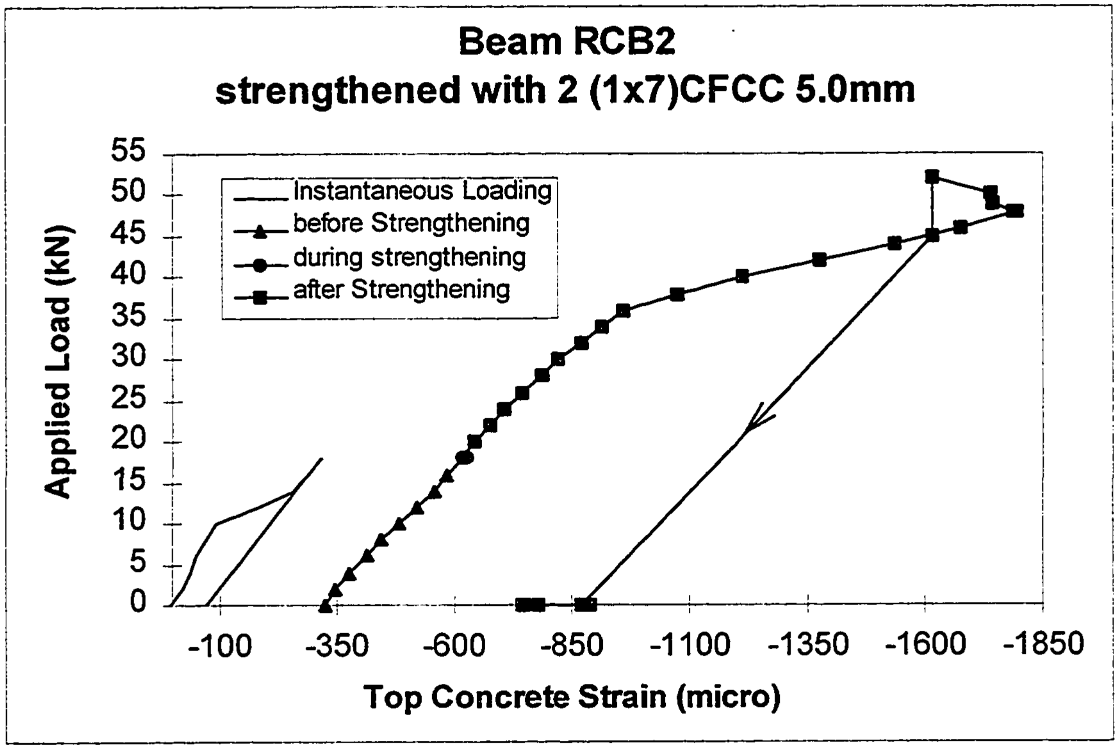


Figure 4.13a Load versus Top Concrete Strain for Beam RCB2

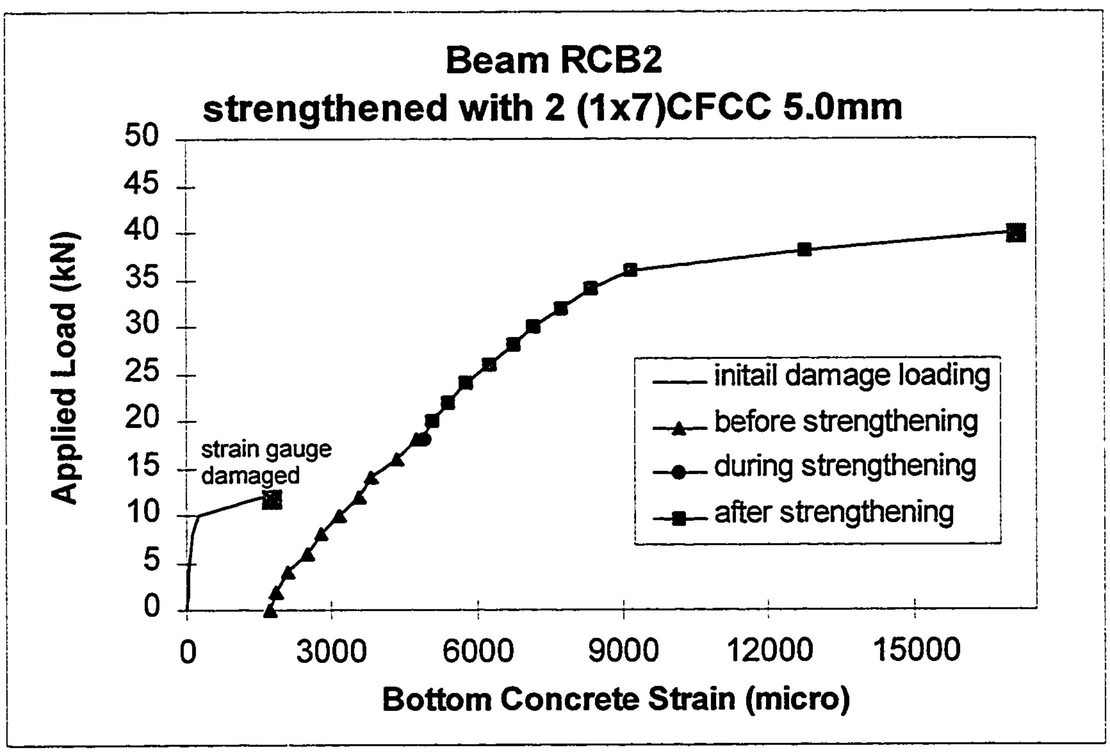


Figure 4.13b Load versus Bottom Concrete Strain for Beam RCB2

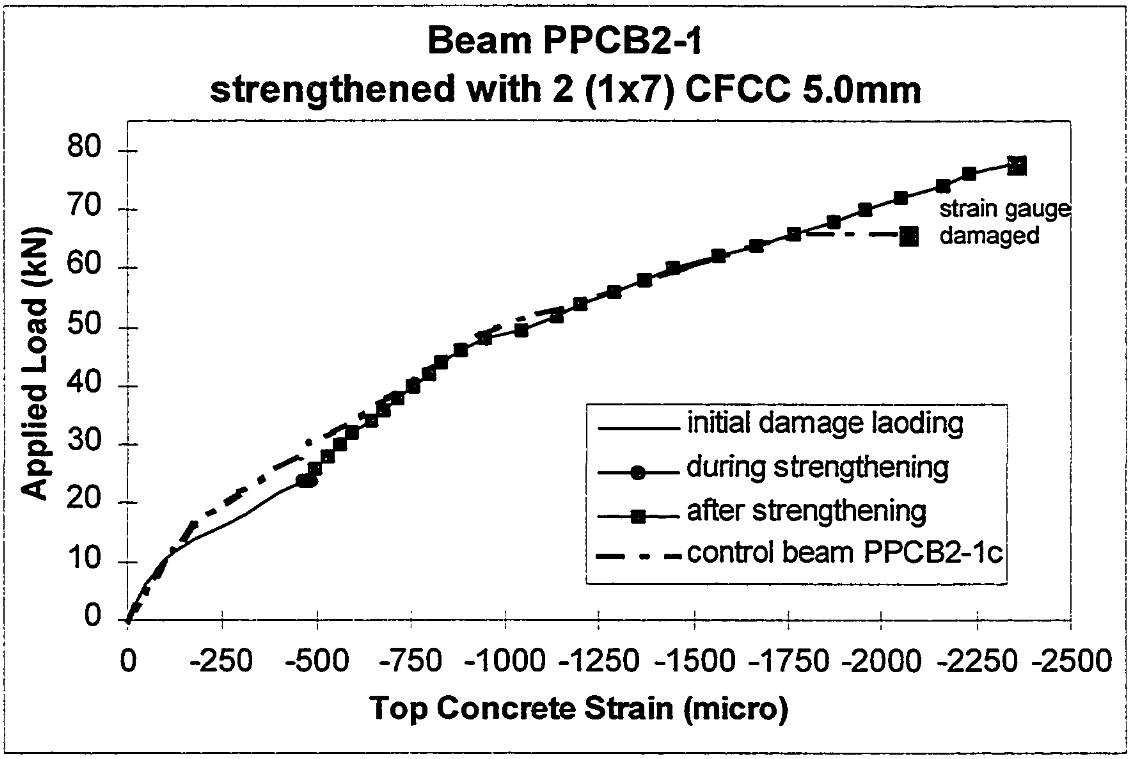


Figure 4.14a Load versus Top Concrete Strain for Beam PPCB2-1

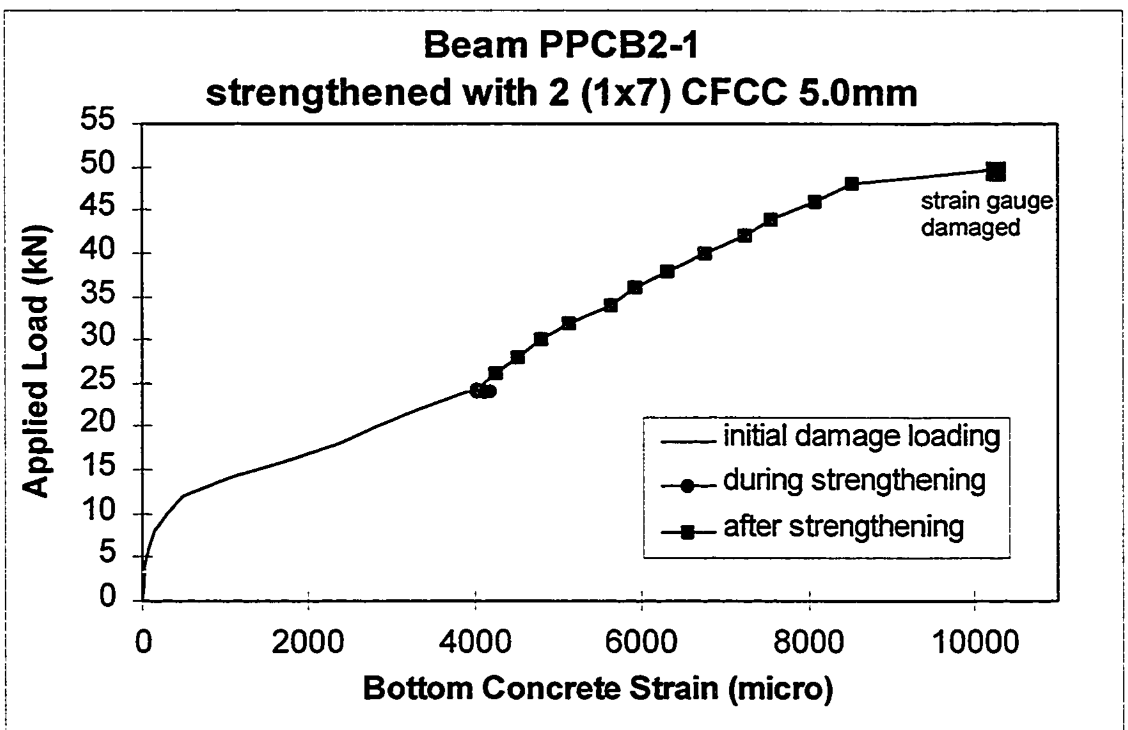


Figure 4.14b Load versus Bottom Concrete Strain for Beam PPCB2-1

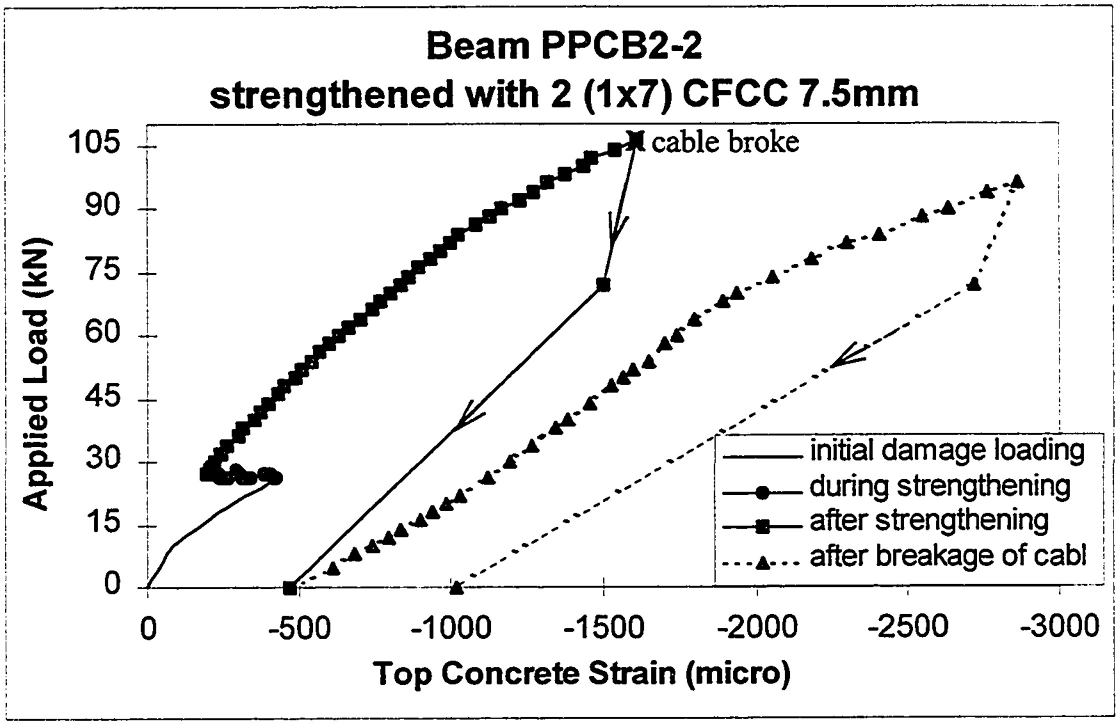


Figure 4.15a Load versus Top Concrete Strain for Beam PPCB2-2

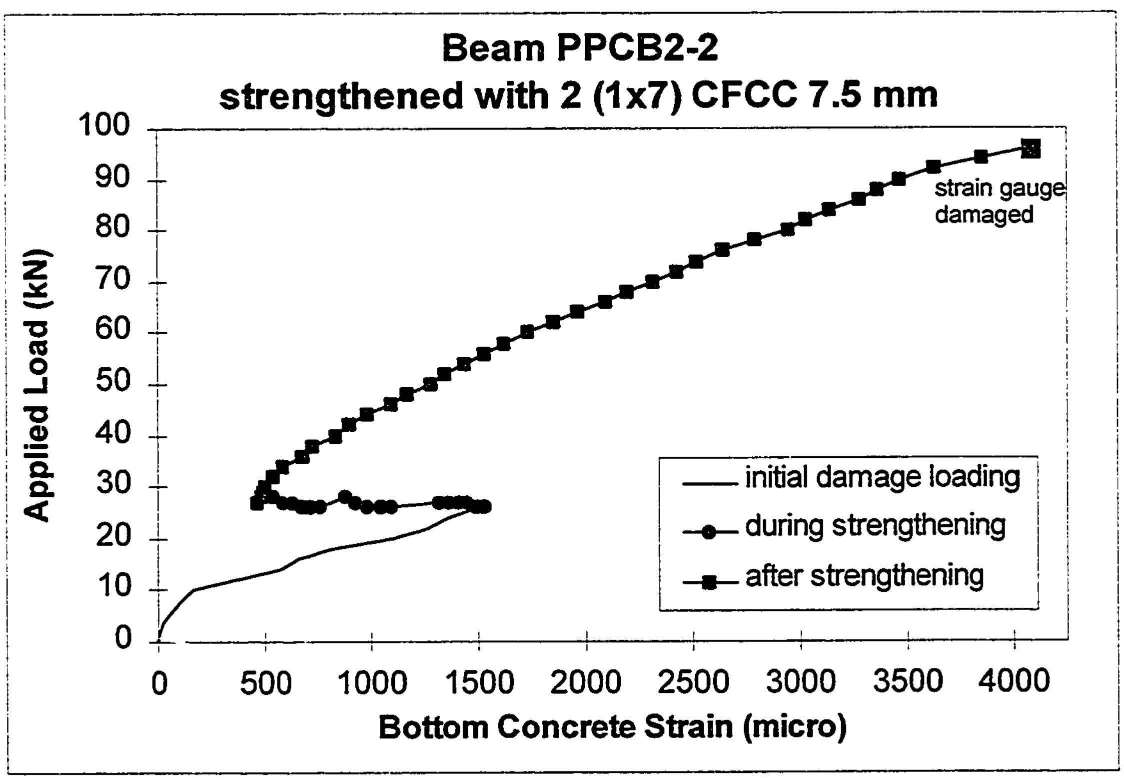


Figure 4.15b Load versus Bottom Concrete Strain for Beam PPCB2-2

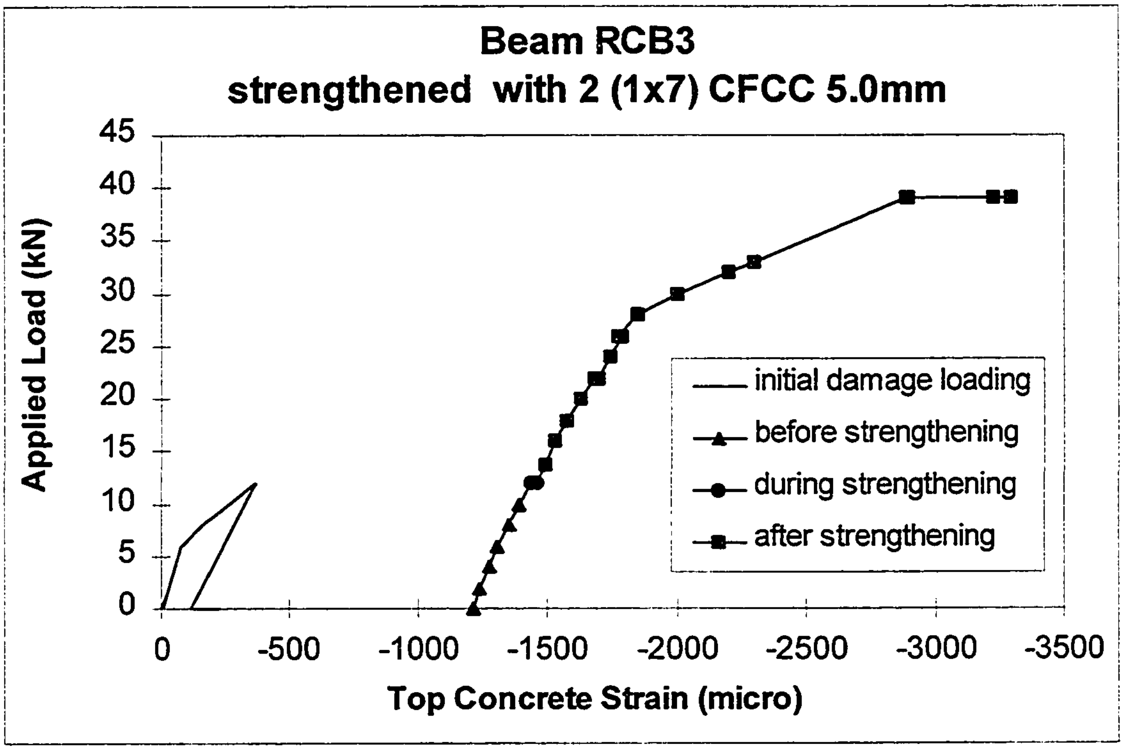


Figure 4.16a Load versus Top Concrete Strain for Beam RCB3

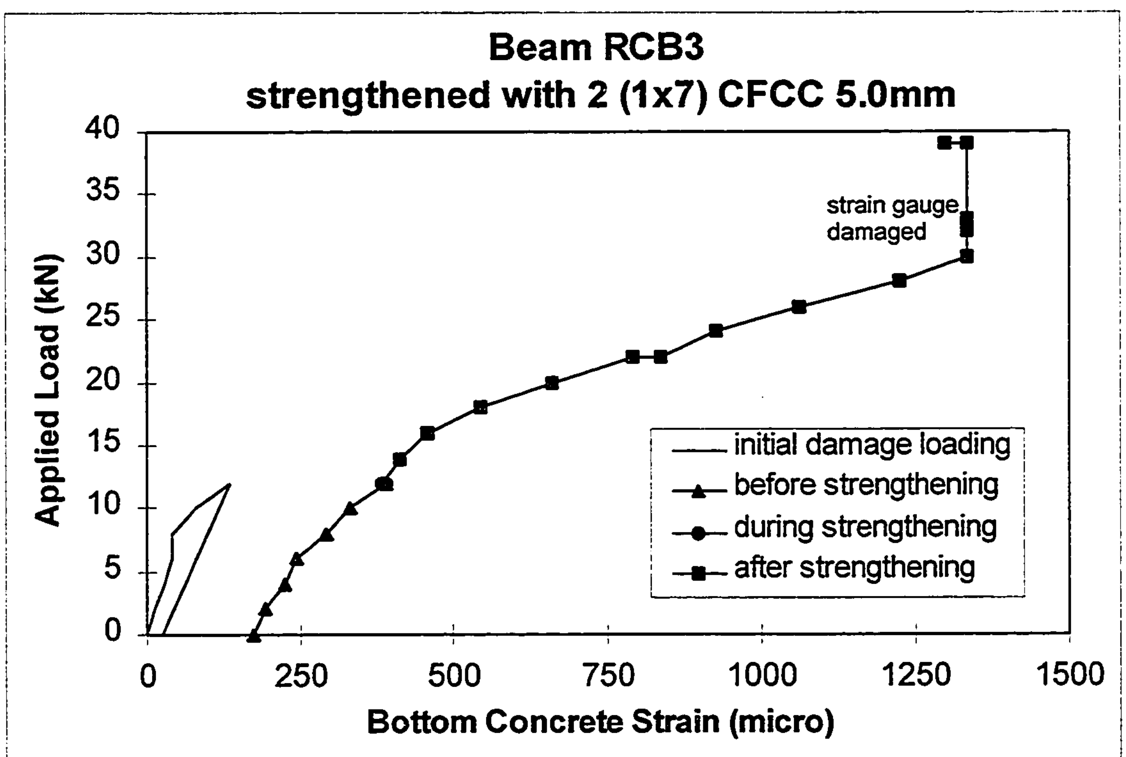


Figure 4.16b Load versus Bottom Concrete Strain for Beam RCB3

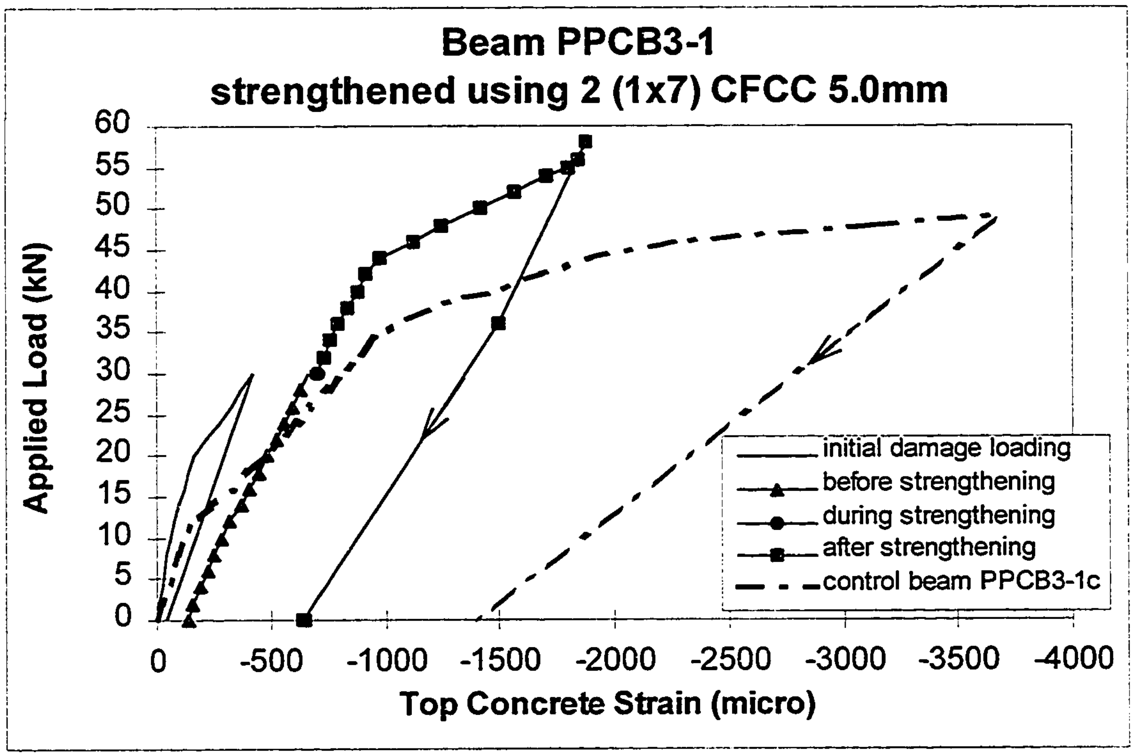


Figure 4.17 Load versus Top Concrete Strain for Beam PPCB3-1

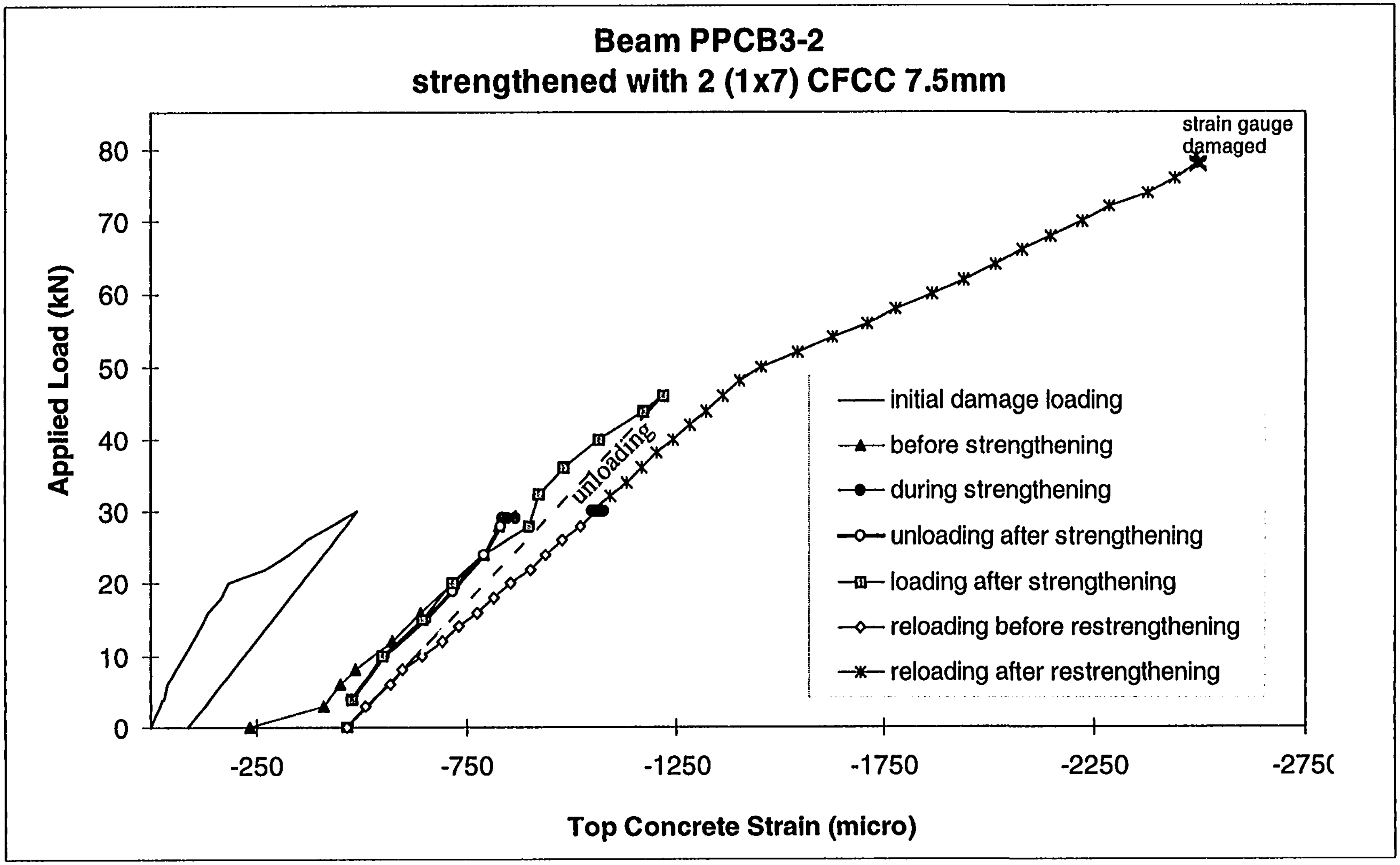


Figure 4.18 Load versus Top Concrete Strain for Beam PPCB3-2

4.5 Stresses in the Ordinary Tension Steel

The stresses in the ordinary tension steel were calculated from the measured strains using strain gauges attached to the surface of the reinforcing bars at the member midspan location. Variation of the reinforcing steel strain with the applied load over the entire loading history before and after external prestressing is shown in Figures 4.19 to 4.23 for beams PPCB1-1, PPCB1-2, PPCB2-2, RCB3, and PPCB3-1 as well as for the control beams PPCB1-1c and PPCB3-1c. The strain gauges in beams RCB1, RCB2, PPCB2-1, PPCB3-2 and PPCB2-1c failed during the tests and therefore, the strain measurements in these beams were not possible. Yielding of the ordinary reinforced steel occurred in all partially prestressed specimens except PPCB1-1, and PPCB1-1c as can be seen in the load-steel strain response. The load at which yielding occurred and the yield stress of the reinforcing steel was marked by further increase in deformation and in a rapid increase in the strain gauge readings and sudden flattening of the load-deflection response. Because of the excessive crack widths that normally accompany the yield process of reinforcing bars, the strain gauges attached to the reinforcing bars in some beam specimens either failed or gave unreliable readings shortly after yielding. This explains the missing data of the strain in the reinforcing bars after yielding of some specimens.

As shown in the figures, the increase in strain in the reinforcing steel with applied load before and after external prestressing is quite similar to the load deflection response.

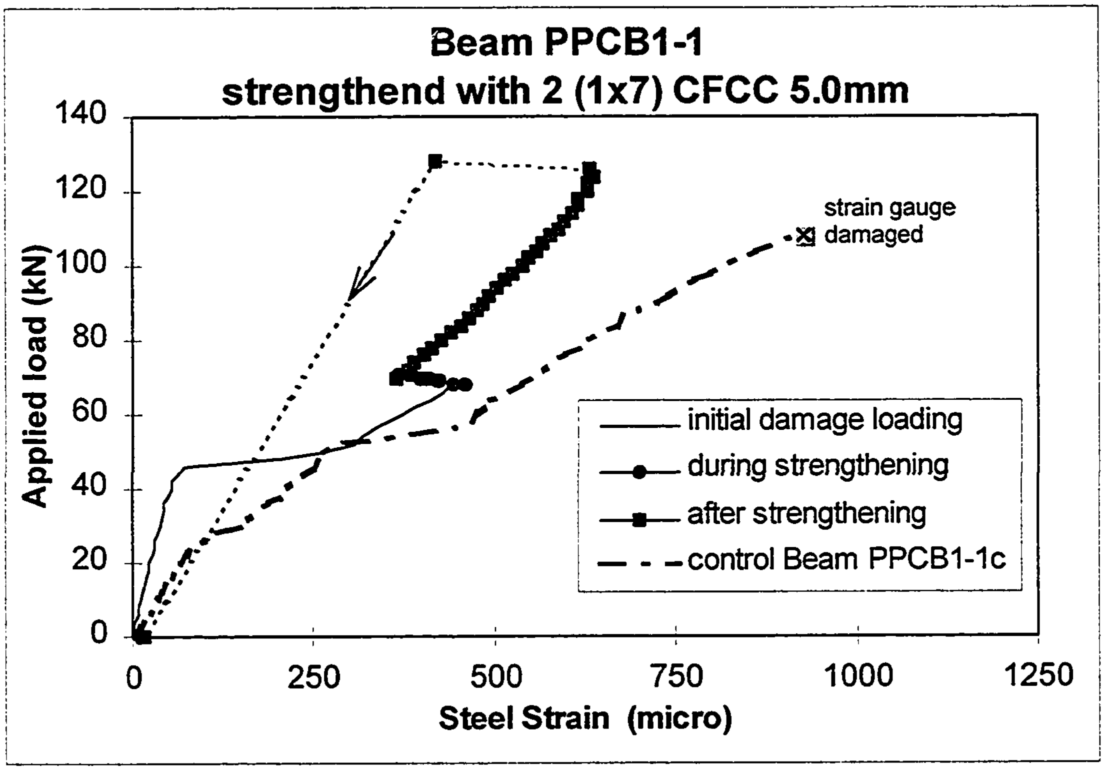


Figure 4.19 Load versus Steel Strain for Beam PPCB1-1

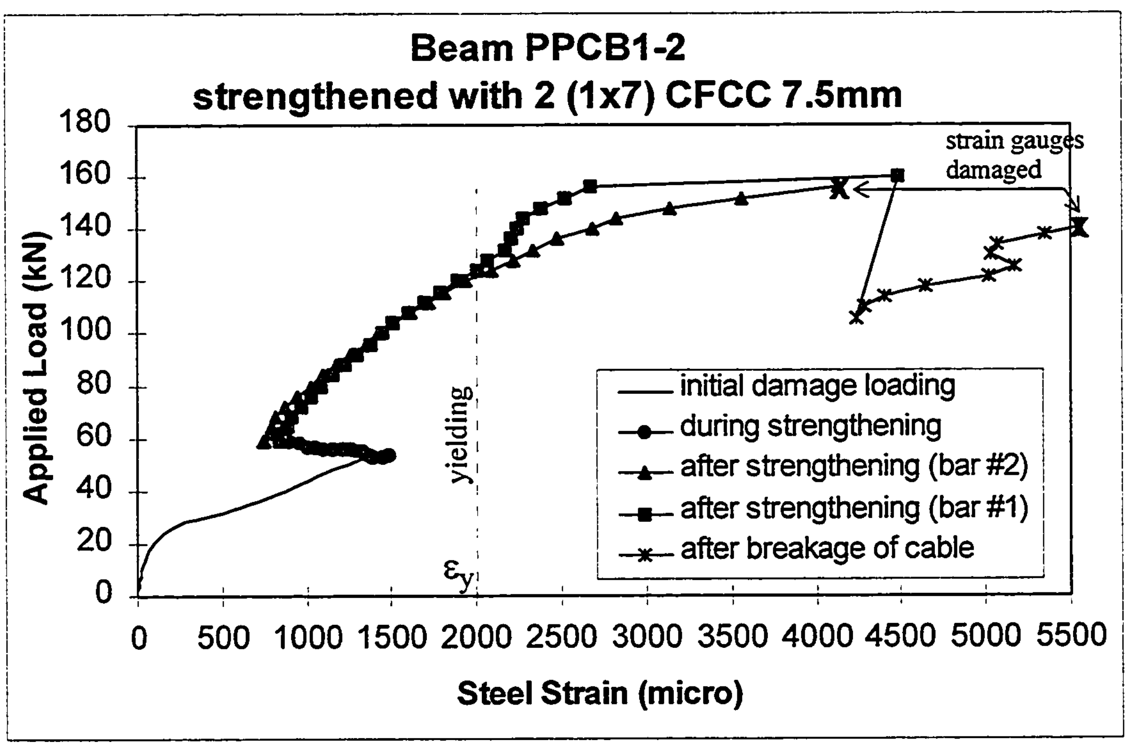


Figure 4.20 Load versus Steel Strain for Beam PPCB1-2

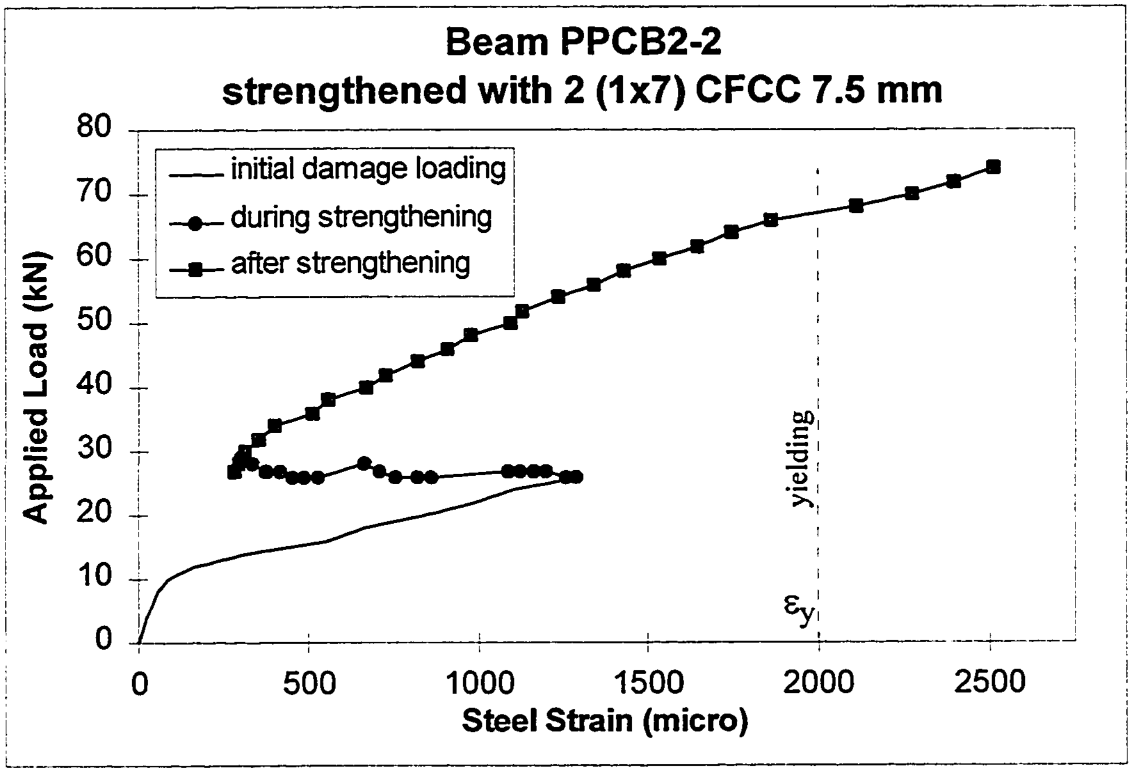


Figure 4.21 Load versus Steel Strain for Beam PPCB2-2

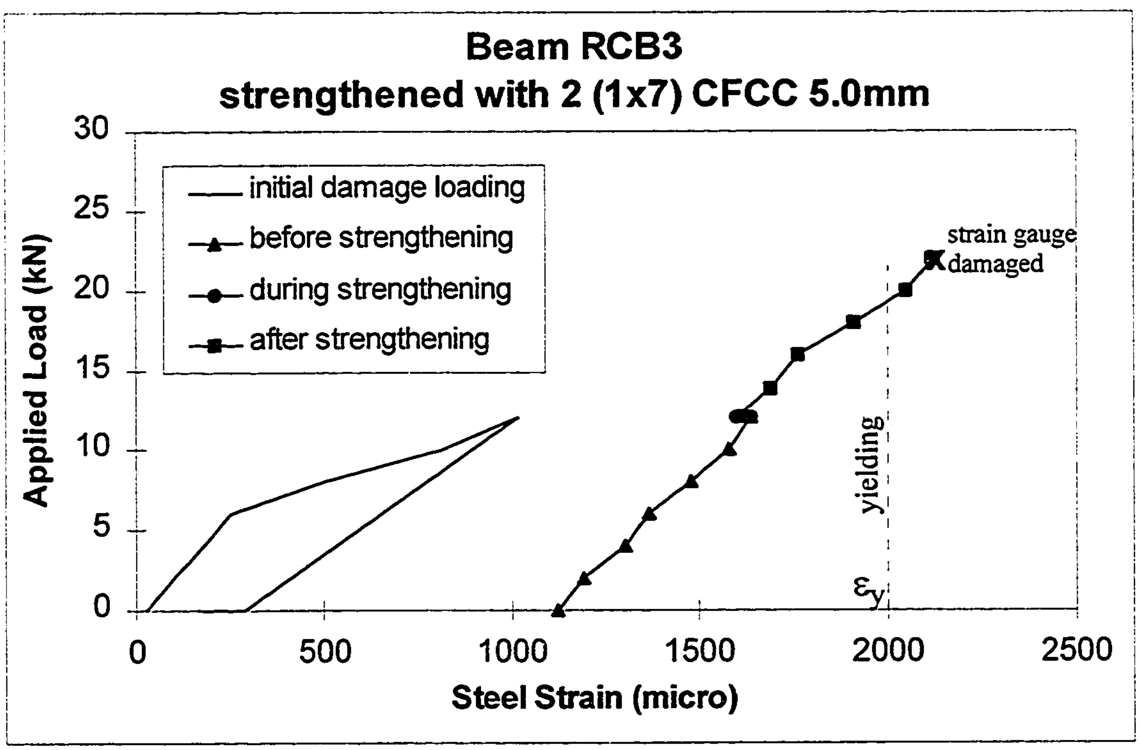


Figure 4.22 Load versus Steel Strain for Beam RCB3

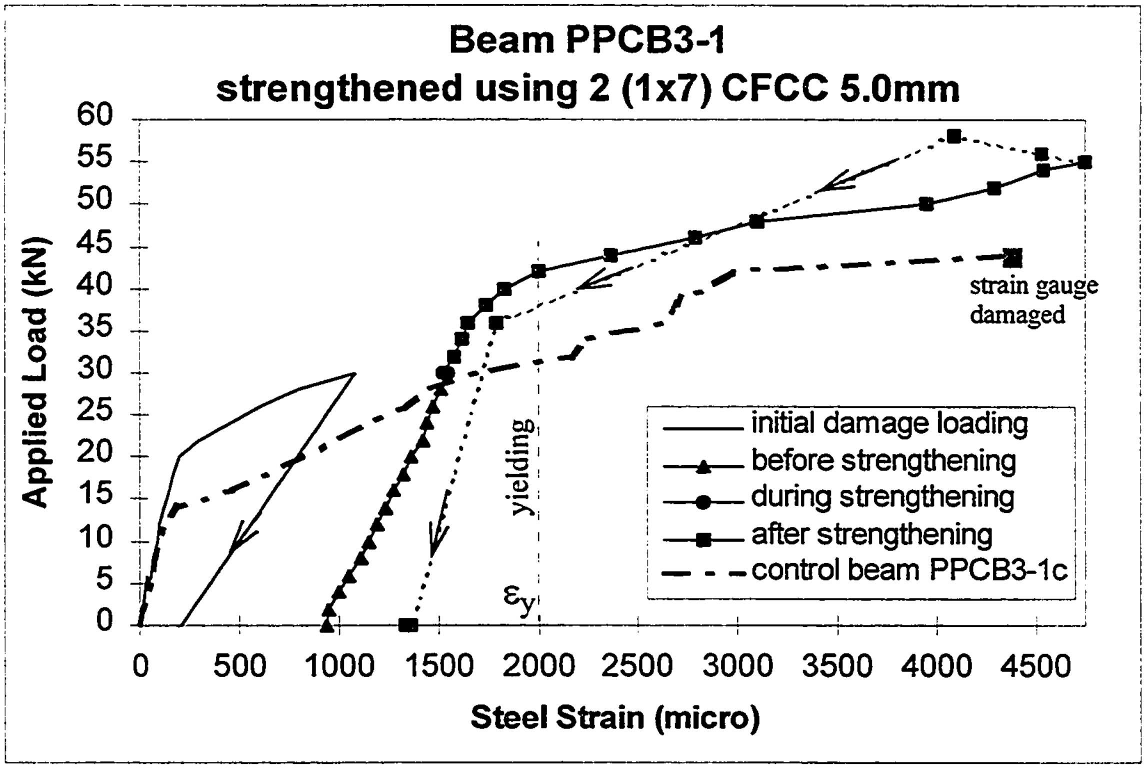


Figure 4.23 Load versus Steel Strain for Beam PPCB3-1

4.6 Force in the External Cables

The variation of the measured force in the external prestressing CFCC cables with the applied load over the entire loading history after strengthening is shown for all beams in Figures 4.24 to 4.32. The relation between the measured external prestressing force with the deflection at midspan is shown in Figures 4.33 to 4.41. It should be noted that in beams PPCB1-2 and PPCB2-2, a slip took place in the CFCC cables causing a slight reduction (Figure 4.26 for PPCB1-2) or no change (Figure 4.29 for PPCB2-2) in the prestressing force in the cables under some increments of applied load when the load was increasing after strengthening.

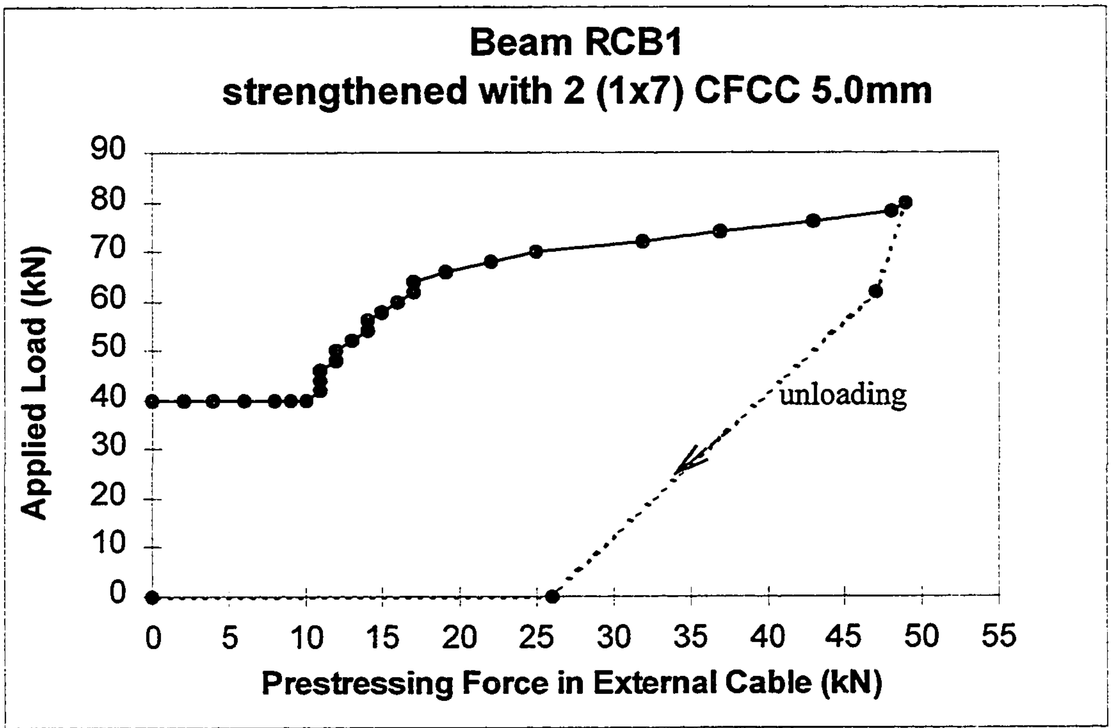


Figure 4.24 Load versus Prestressing Force in External Cable for Beam RCB1

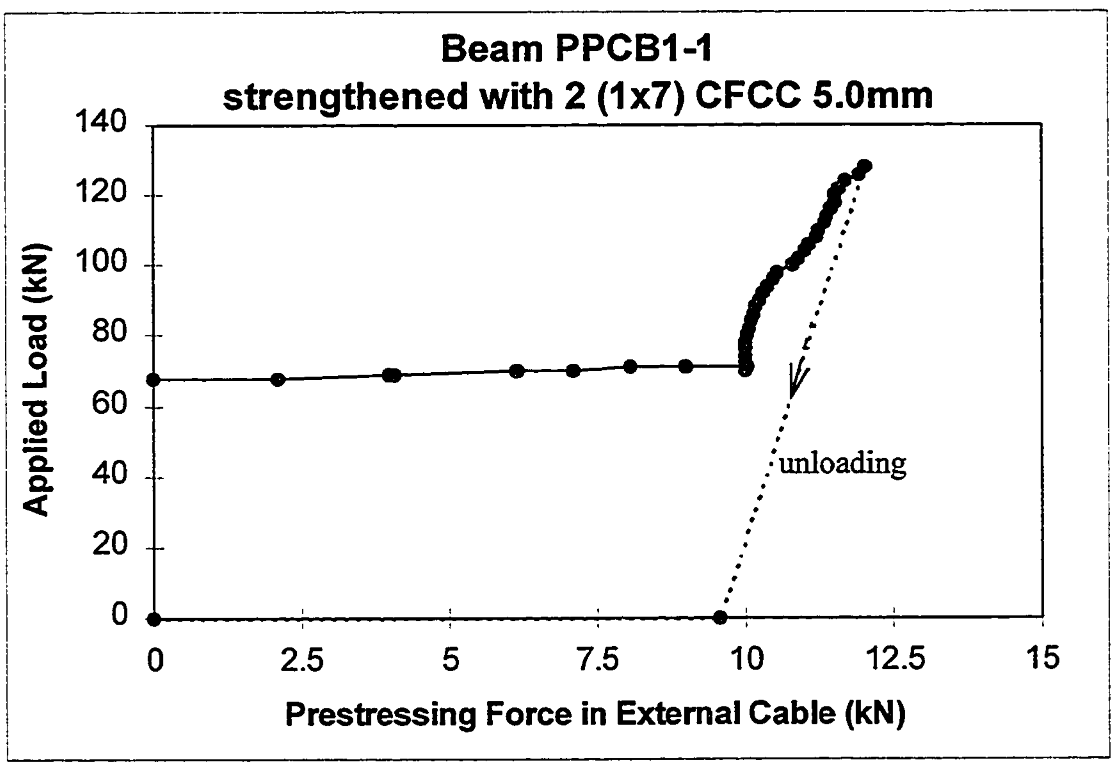


Figure 4.25 Load versus Prestressing Force in External Cable for Beam PPCB1-1

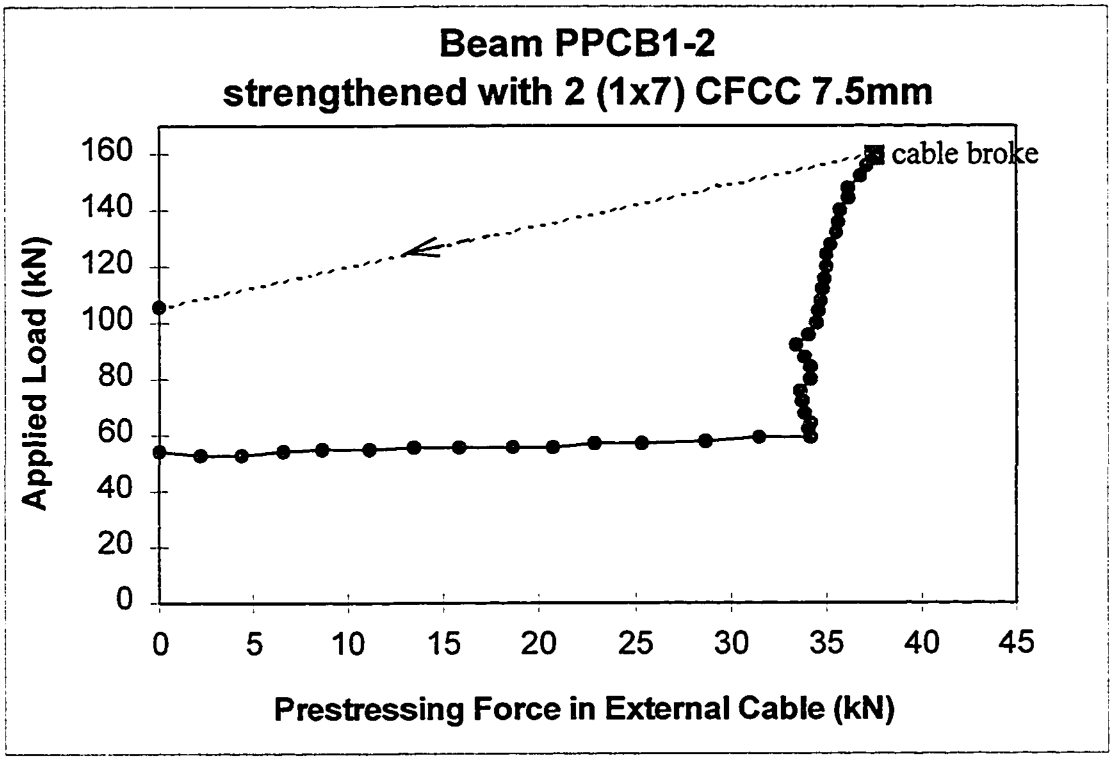


Figure 4.26 Load versus Prestressing Force in External Cable for Beam PPCB1-2

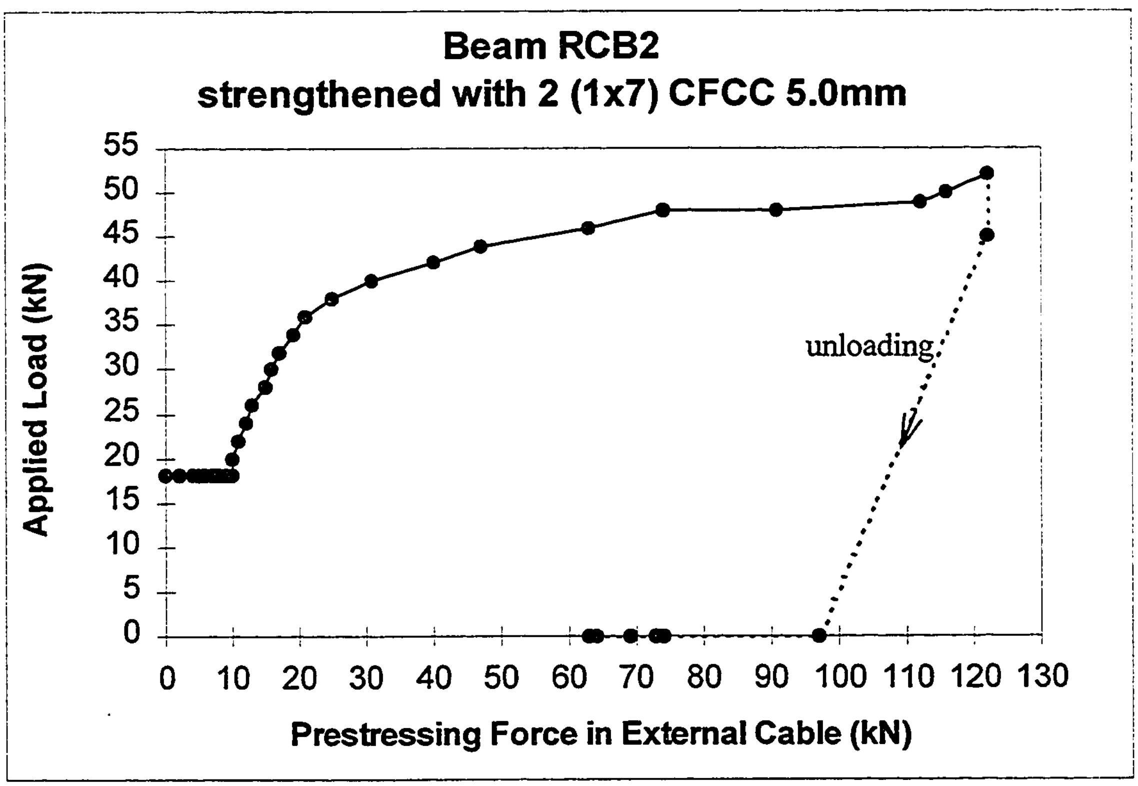


Figure 4.27 Load versus Prestressing Force in External Cable for Beam RCB2

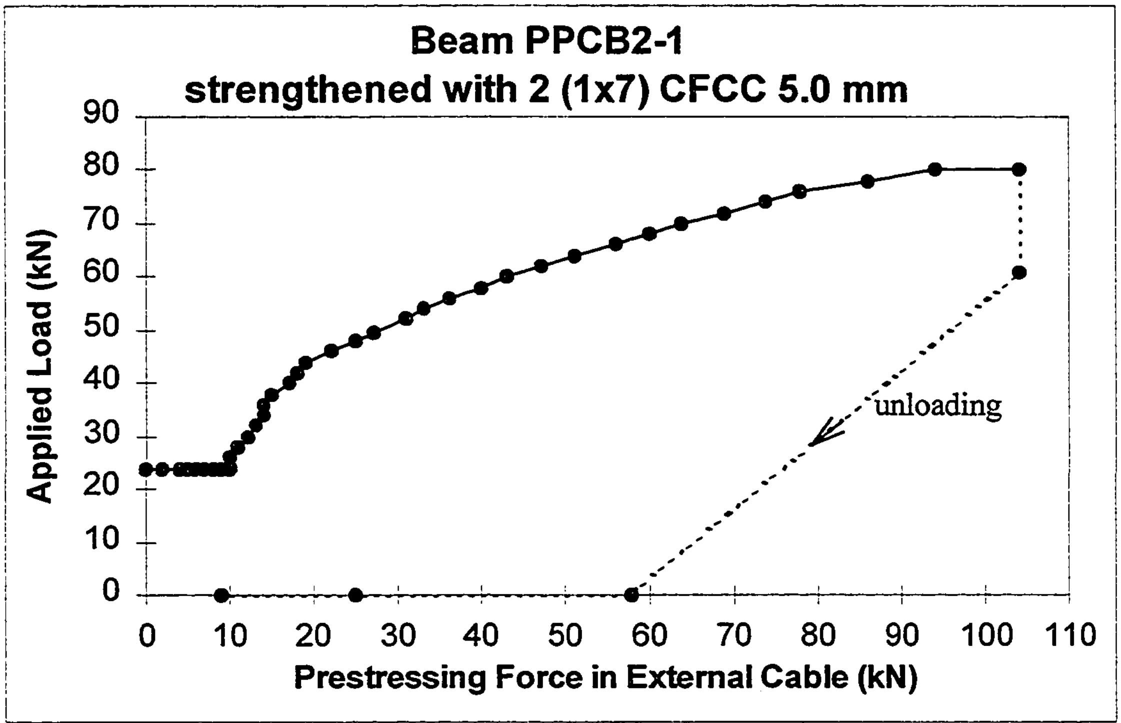


Figure 4.28 Load versus Prestressing Force in External Cable for Beam PPCB2-1

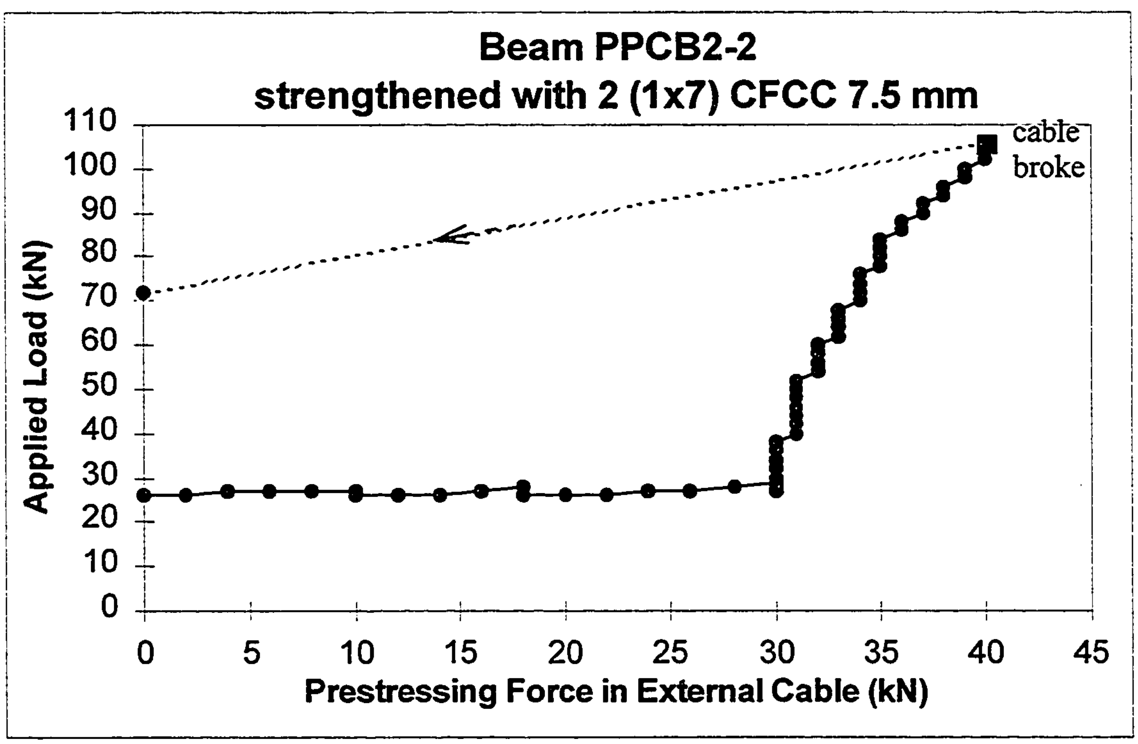


Figure 4.29 Load versus Prestressing Force in External Cable for Beam PPCB2-2

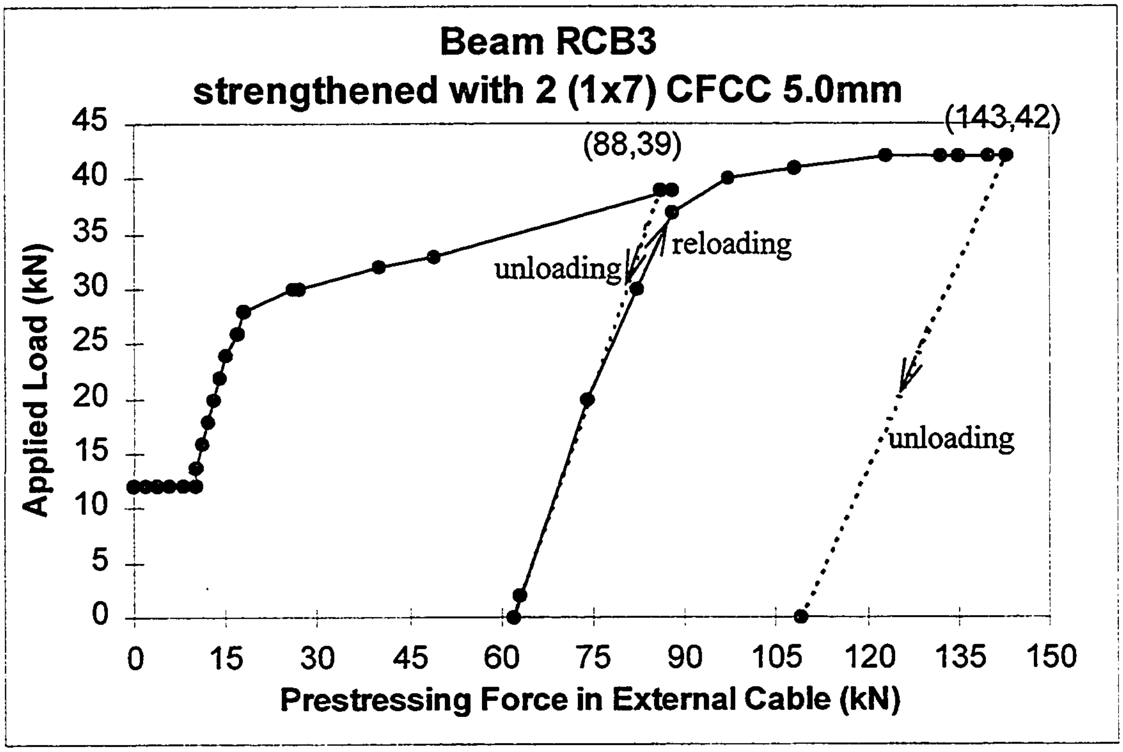


Figure 4.30 Load versus Prestressing Force in External Cable for Beam RCB3

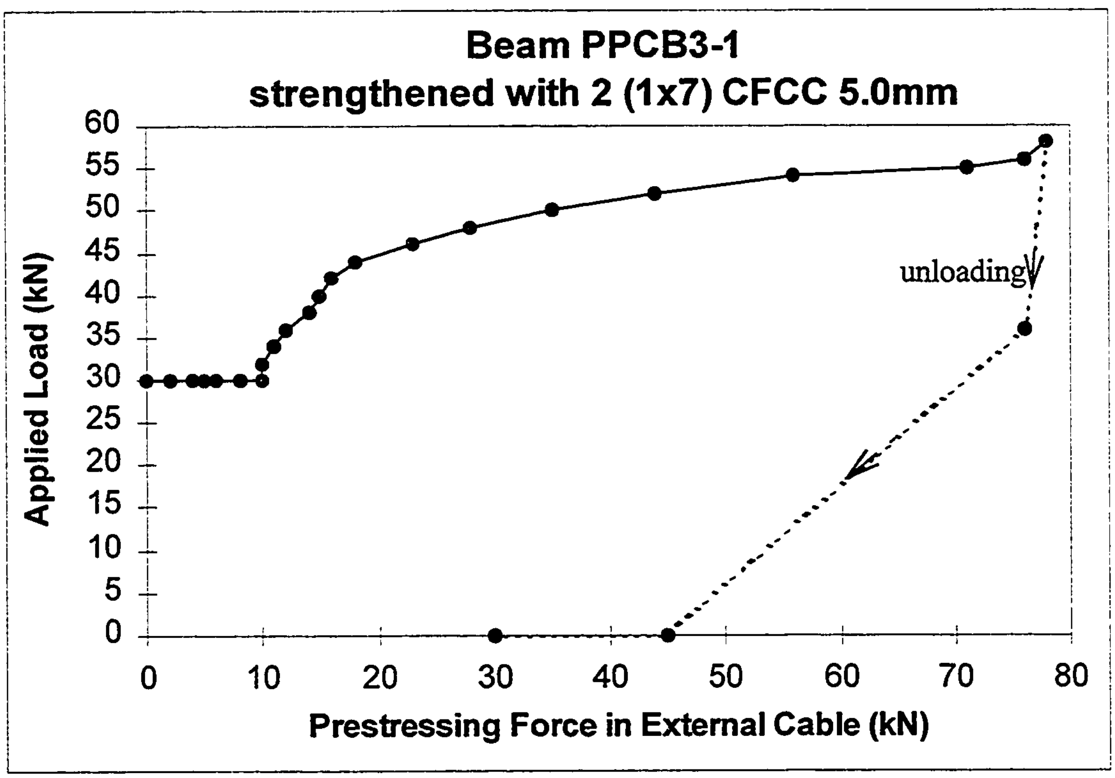


Figure 4.31 Load versus Prestressing Force in External Cable for Beam PPCB3-1

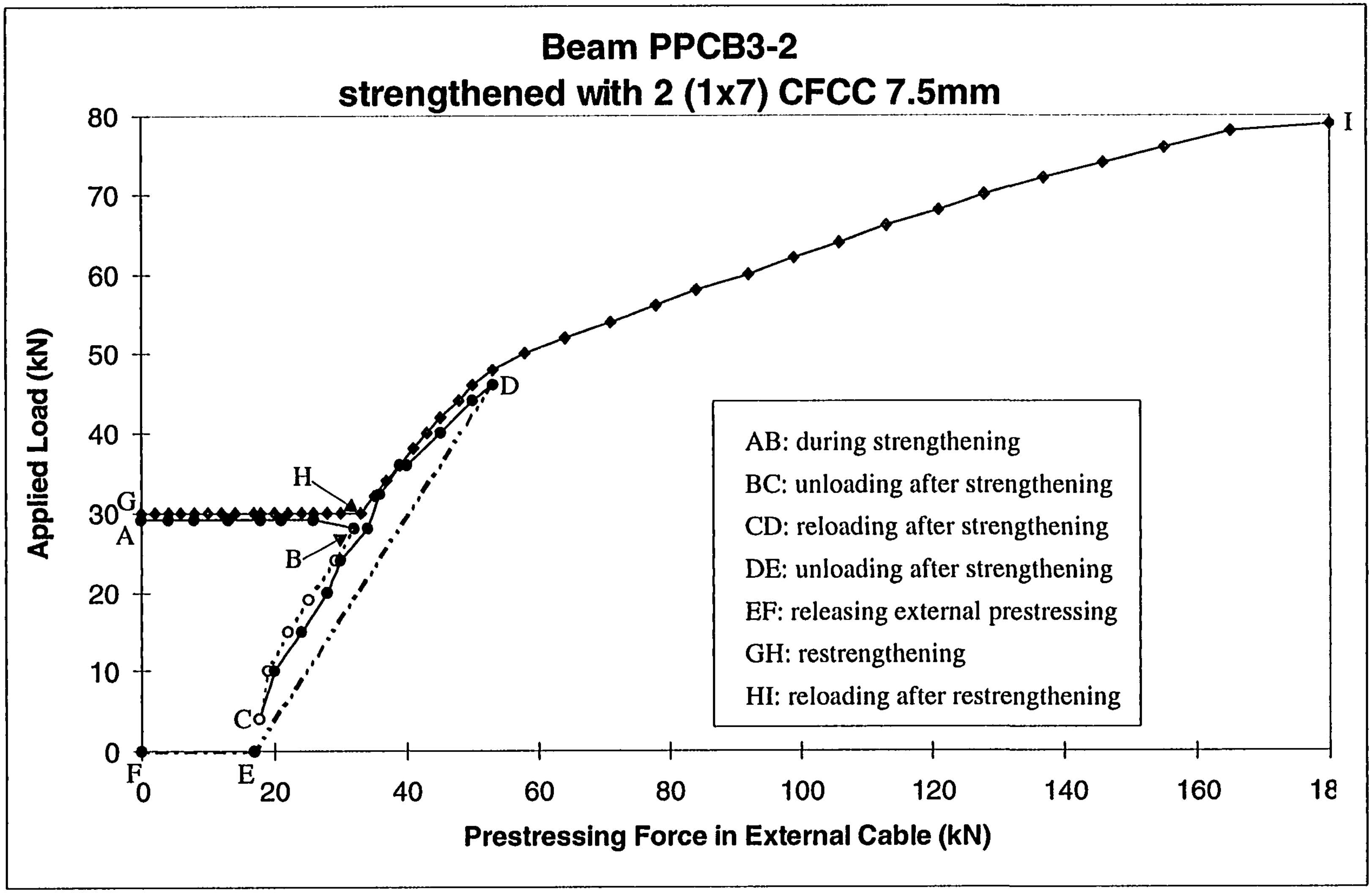


Figure 4.32 Load versus Prestressing Force in External Cable for Beam PPCB3-2

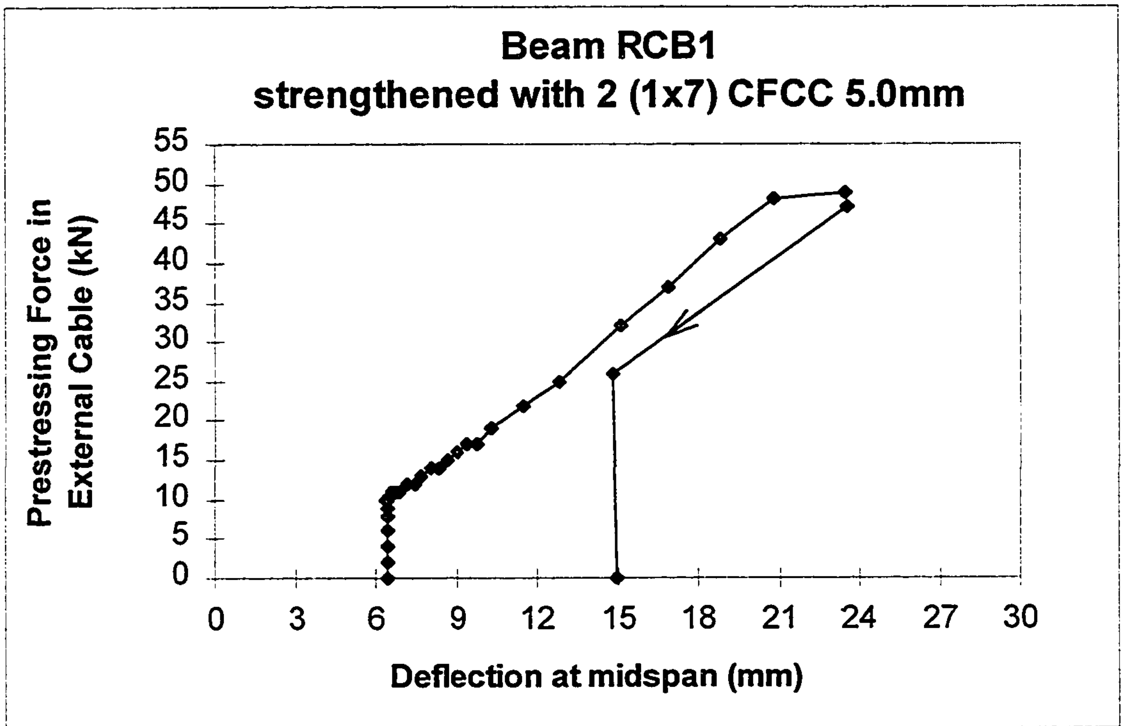


Figure 4.33 Prestressing Force in External Cable versus Deflection for Beam RCB1

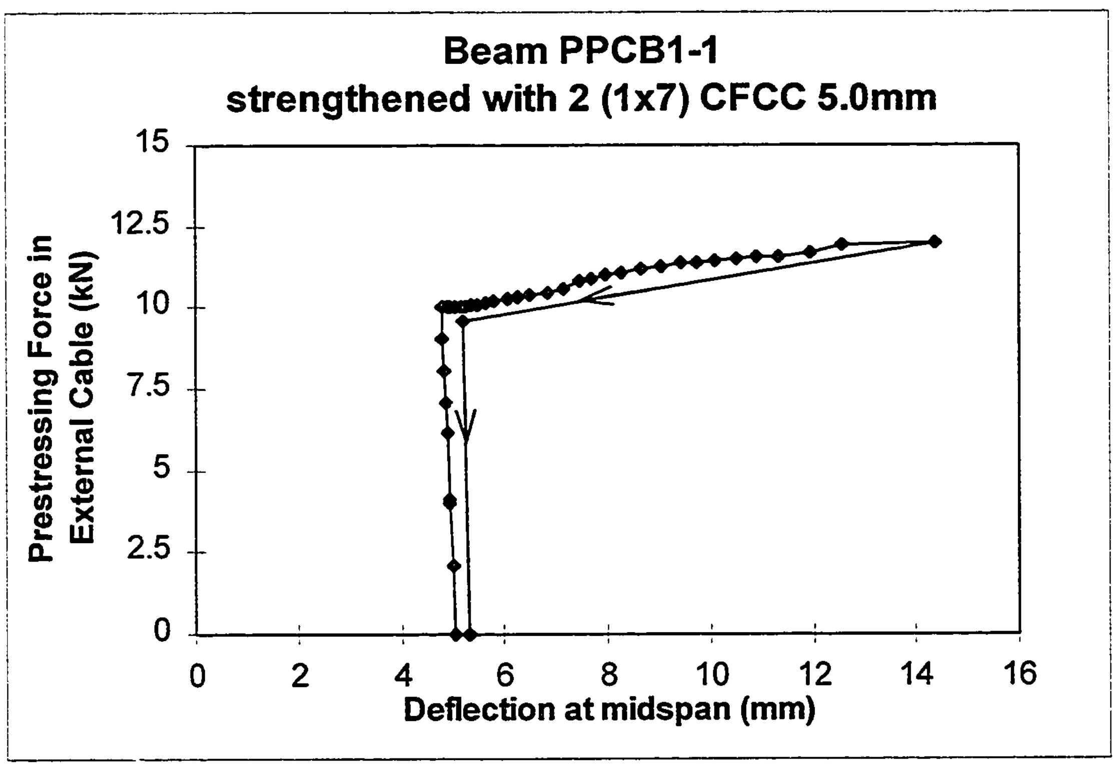


Figure 4.34 Prestressing Force in External Cable versus Deflection for Beam PPCB1-1

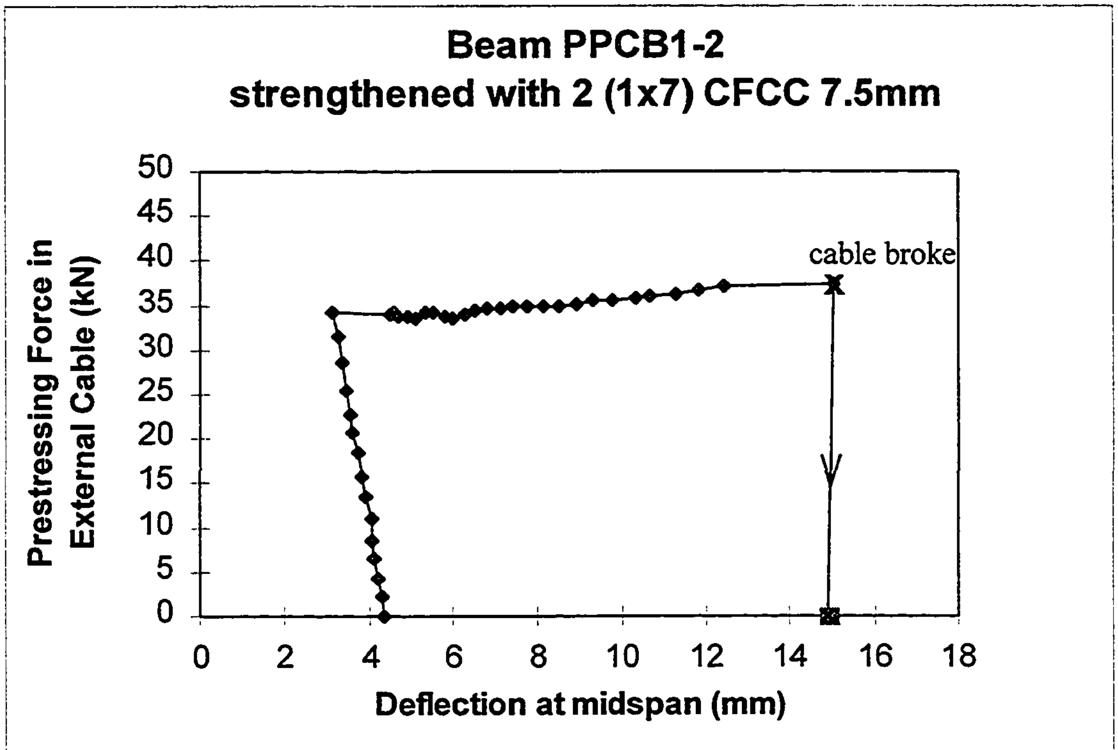


Figure 4.35 Prestressing Force in External Cable versus Deflection for Beam PPCB1-2

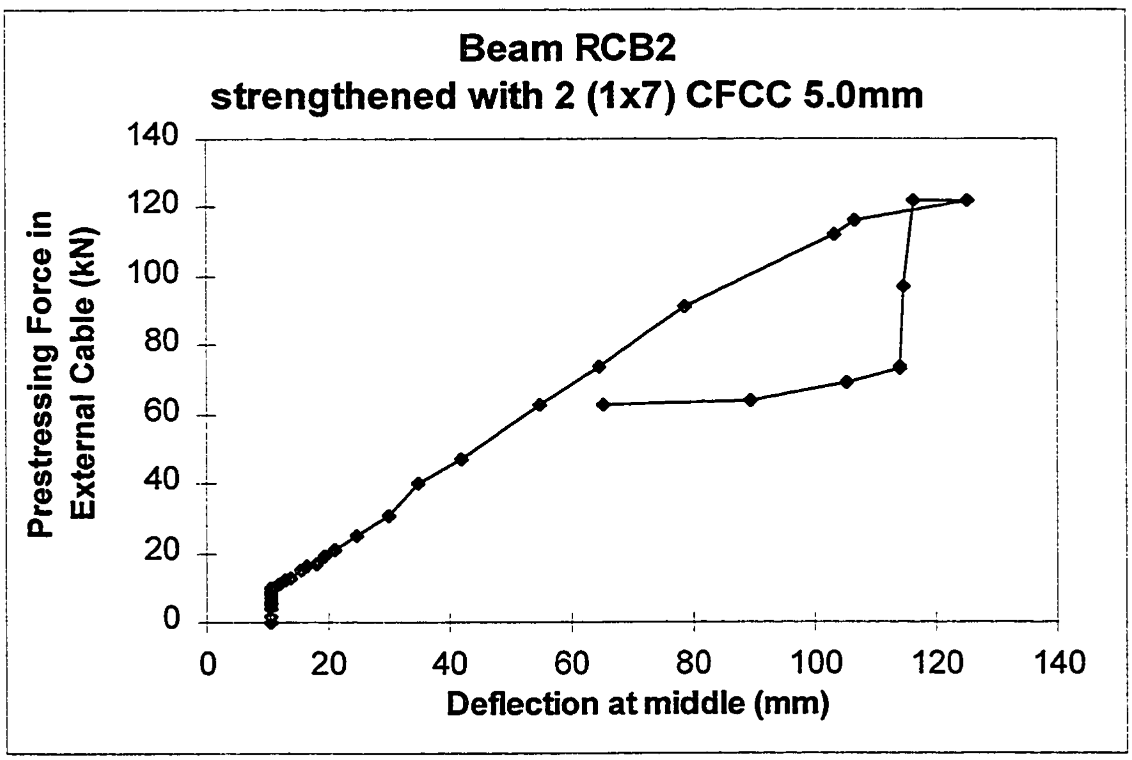


Figure 4.36 Prestressing Force in External Cable versus Deflection for Beam RCB2

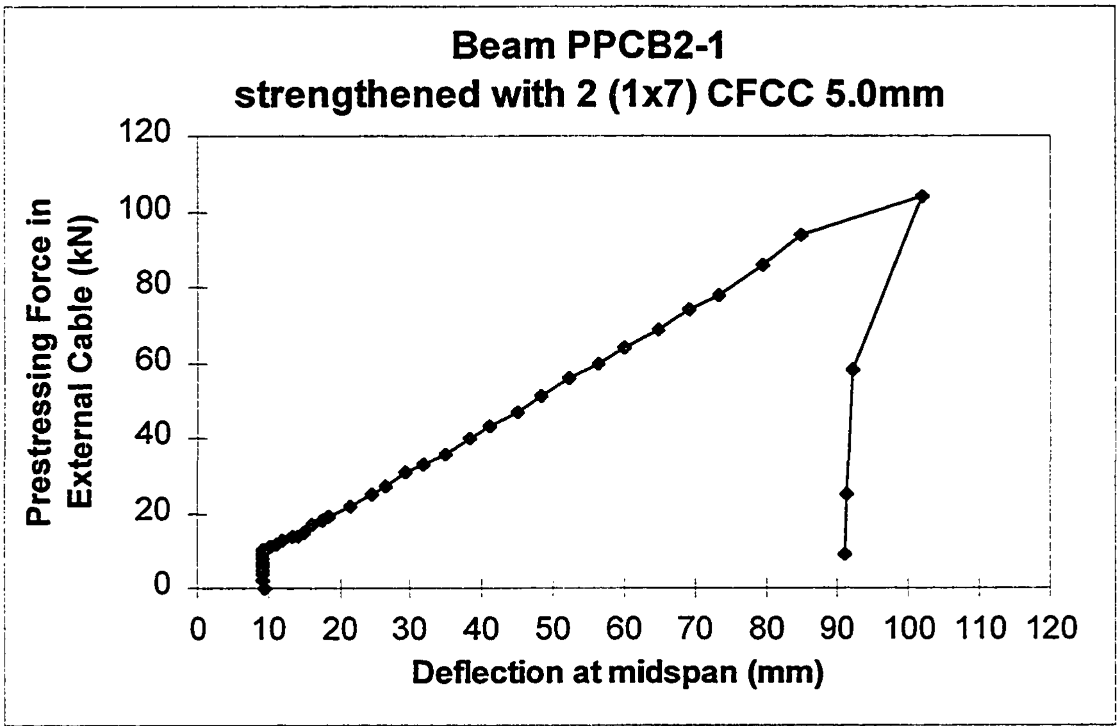


Figure 4.37 Prestressing Force in External Cable versus Deflection for Beam PPCB2-1

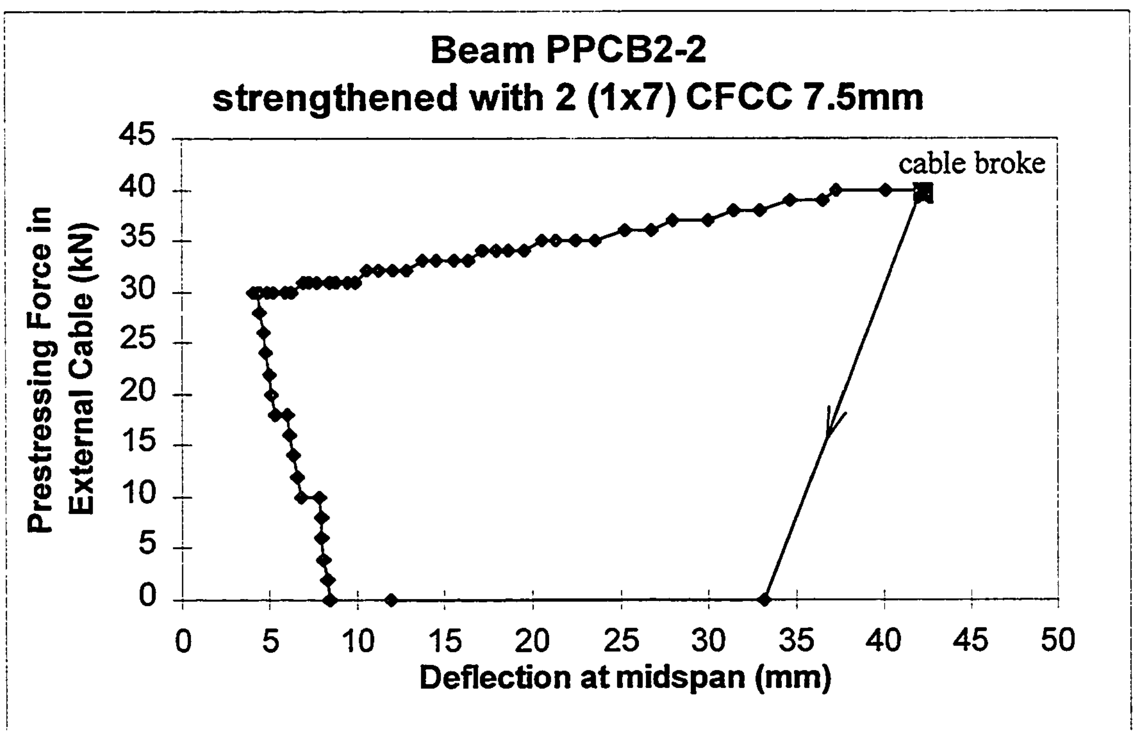


Figure 4.38 Prestressing Force in External Cable versus Deflection for Beam PPCB2-2

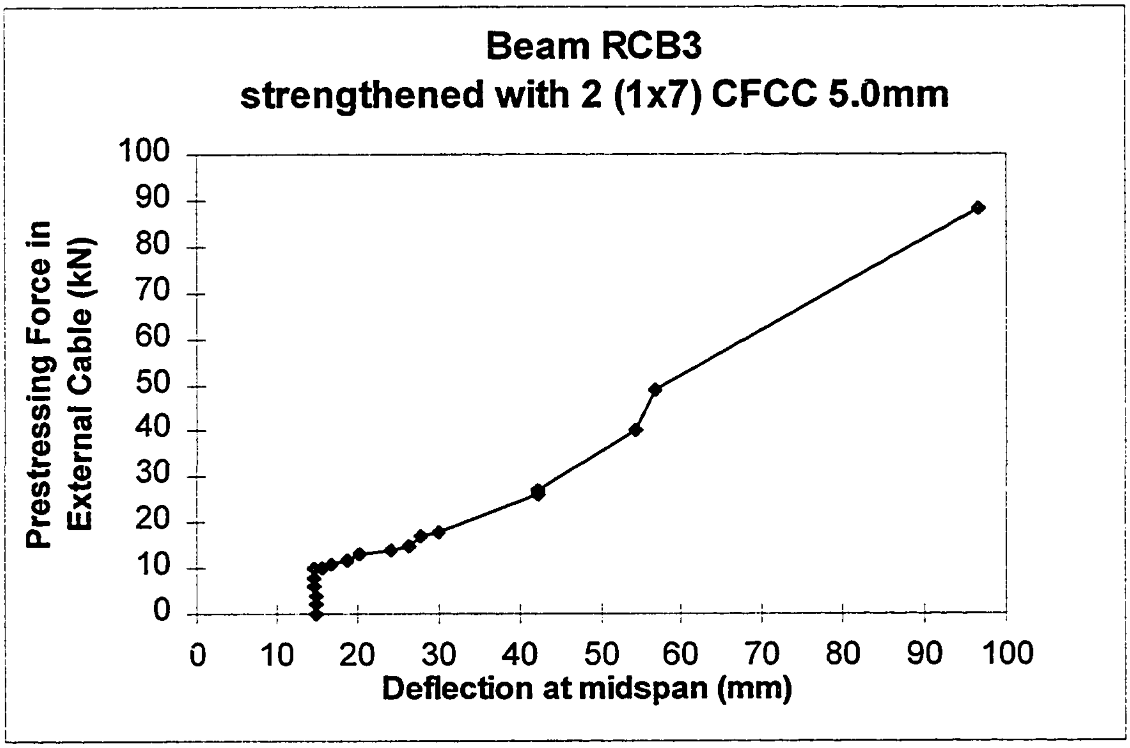


Figure 4.39 Prestressing Force in External Cable versus Deflection for Beam RCB3

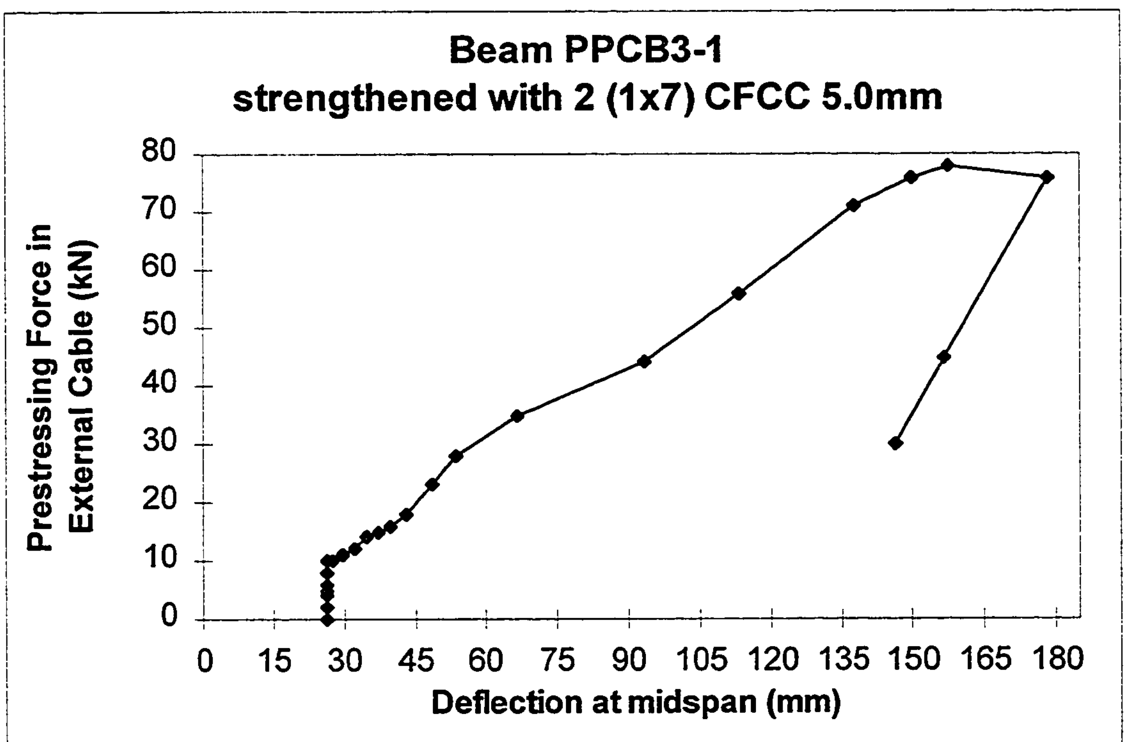


Figure 4.40 Prestressing Force in External Cable versus Deflection for Beam PPCB3-1

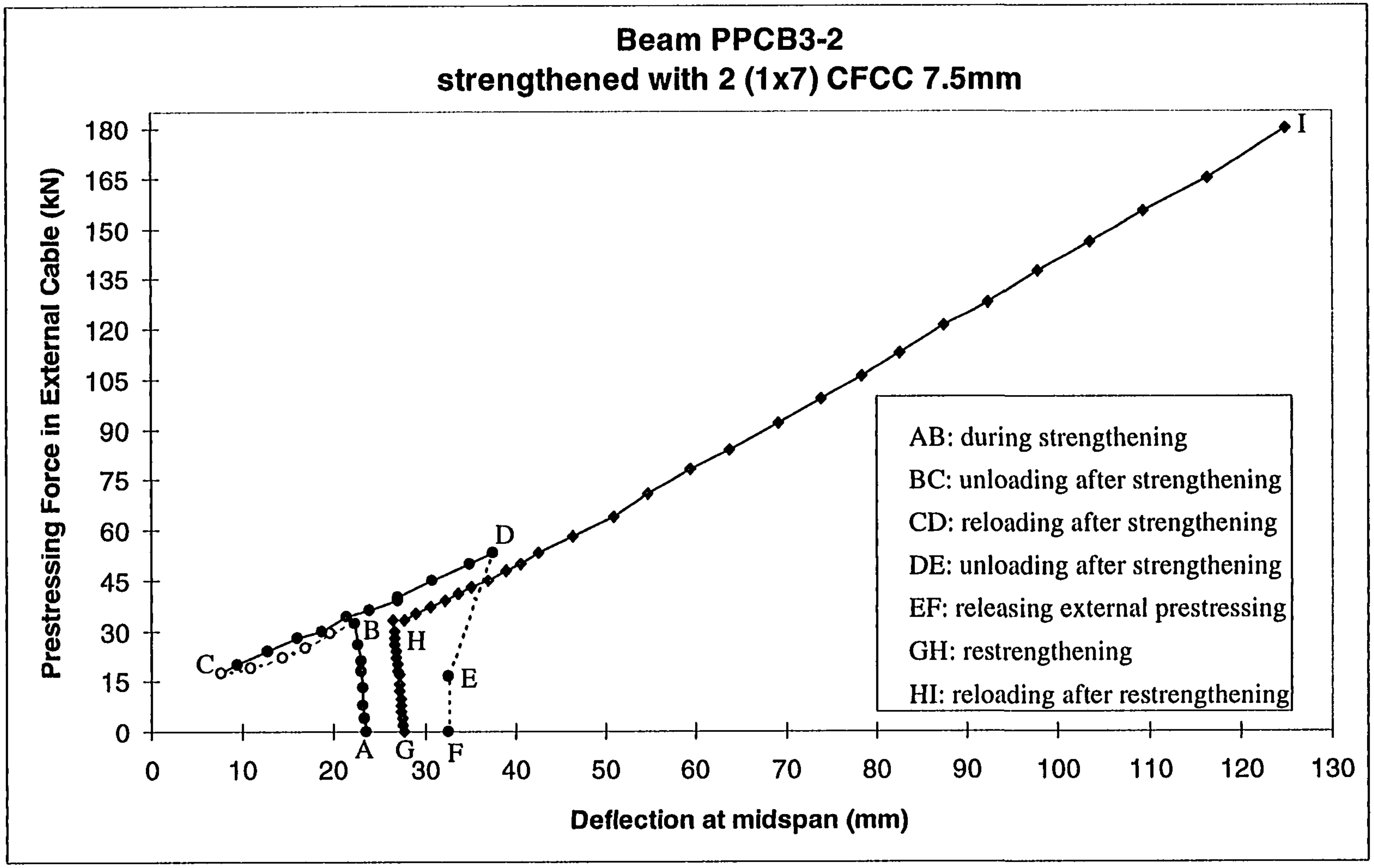


Figure 4.41 Prestressing Force in External Cable versus Deflection for Beam PPCB3-2

4.7 Ultimate Capacity without Strengthening

In order to check the ultimate moment capacity of the beams without strengthening, the strain compatibility method has been used. This method is more accurate in predicting the ultimate moment capacity of the beam than any other method. Calculation of this capacity was useful in determining a lower bound of the required strength of the load frame and capacity of the jacks for laboratory testing. It should be noted that no reduction capacity factor was used in the theoretical ultimate moment capacity calculations. Table 4.1 shows the values of calculated ultimate loads of unstrengthened beams. The stress in the internal bonded prestressing steel at the ultimate flexural strength was calculated using the approximate values of f_{ps} given by Eq. (18-1) in the CSA Standard A23.3-94 Code based on strain compatibility. For members with bonded tendons:

$$f_{ps} = f_{pu}\left(1 - k_p \frac{c}{d_p}\right) \text{ MPa} \tag{4.1}$$

provided $\frac{c}{d_p} \leq 0.5$ and $f_{pe} > 0.6 f_{py}$

where:

$$k_p = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \tag{4.2}$$

the term k_p accounts for the different slopes of the stress- strain curves for the different types of prestressing steel. The neutral axis depth, c is calculated by assuming a stress f_{pu} in the prestressed steel and a stress f_y in the nonprestressed reinforcement. It should be noted that c can be computed from equilibrium of force in the section. For a rectangular cross section (Figure 4.44):

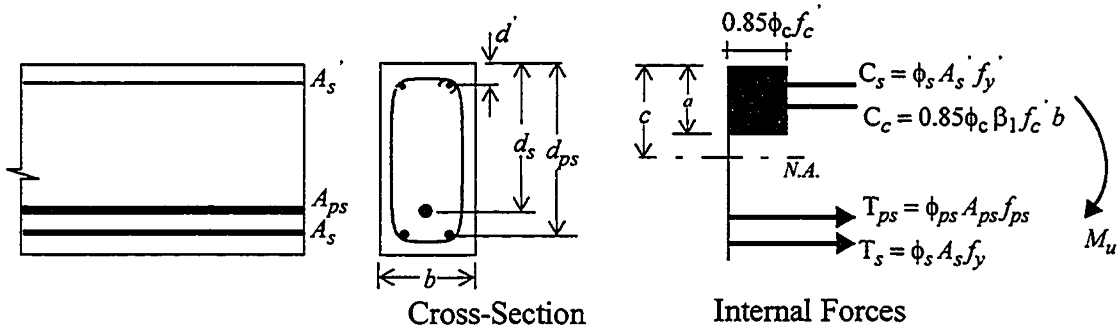
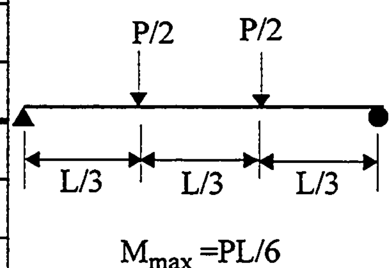


Figure 4.42 Internal Forces in Rectangular Cross-Section

Table 4.1: Theoretical Ultimate Capacity of Unstrengthened Beams

Beam Designation	$P_{ultimate}$ (kN)
RCB1	57.44
PPCB1-1	108.43
PPCB1-2	153.76
RCB2	32.82
PPCB2-1	61.96
PPCB2-2	87.86
RCB3	22.00
PPCB3-1	44.00
PPCB3-2	61.50



From equilibrium of internal forces: $C_c = T_{ps} + T_s - C_s$

$$0.85\phi_c f_c' ba = \phi_p A_{ps} f_{ps} + \phi_s A_s f_y - \phi_s A_s' f_y' \tag{4.3}$$

where $a = \beta_1 \cdot c$ (4.4)

and $\beta_1 = 0.85 - 0.008[f_c'(\text{MPa}) - 30\text{MPa}] \geq 0.65$ (4.5)

The ultimate flexural strength were calculated using the following equation:

$$M_u = \phi_p A_{ps} f_{ps}\left(d_{ps} - \frac{a}{2}\right) + \phi_s A_s f_y \left(d_s - \frac{a}{2}\right) - \phi_s A_s' f_y \left(\frac{a}{2} - d'\right) \tag{4.6}$$

4.8 Ultimate Flexural Resistance after Strengthening

In all beam specimens, the ultimate load capacity was taken as the peak load when the beam collapsed. The cable profile for external prestressing in all beams was one with a single draped point in the middle. With the increase in the applied load, the beams deflected downward and the eccentricity of the cable from the beam axis decreased between the point of the saddle and the anchorage. Flexural failure was observed in all beams (except in the group of small span, 2.25m) at the section under the point of application of the load. In some beams, the flexural failure propagated in the constant moment region.

A comparison between the calculated ultimate flexural strength of the beam specimens without external prestressing and the observed flexural strength with external prestressing during the test is presented in Tables 4.2 to 4.4. As shown in these tables, the increase in the ultimate flexural resistance of the various beam specimens due to external prestressing varied between a minimum of 4% and maximum of 70%, depending on the span-to-depth ratio, the content of internal tension reinforcement in the specimens and on the level of external prestressing which was not taken as a variable parameter in this investigation.

Table 4.2: Critical Results of Tests on Beams of Group 1 (S/d_p = 10.7) and their Modes of Failure

Beam Designation	*PPR*	ω	Cracking Load (kN)		Ultimate Load (kN)			Deflection at ultimate (mm)	Force in CFCC cable (kN)		Mode of Failure
			Theor.	Exper.	Theoretical without strengthening	Experimental after strengthening	$\frac{P_{exp}}{P_{the}}$ *		initial	at ultimate	
RCB1	0	0.050	22.45	14	57.44	80	1.40	23.6	10	49	S.F.
PPCB1-1	0.53	0.142	44.15	48	108.43	128	1.18	14.39	10	12	S.F.
PPCB1-1c				36		110 no strengthening	1.01	12.50	N/A		S.F.
PPCB1-2	0.70	0.234	66.10	30	153.76	160	1.04	15.01	30	39	cable broke
						154	1.00	25.92		----	beam failed S.F.

Note: N/A: Not Available S.F.: Shear Failure F.F.: Flexural Failure L.C.: Longitudinal Crack

*except for the control beam PPCB1-1c, this ratio indicates the increase in load carrying capacity after strengthening

Table 4.3: Critical Results of Tests on Beams of Group 2 ($S/d_p = 17.8$) and their Modes of Failure

Beam Designation	PPR	ω	Cracking Load (kN)		Ultimate Load (kN)			Deflection at ultimate (mm)	Force in CFCC cable (kN)		Mode of Failure
			Theor.	Exper.	Theoretical without strengthening	Experimental after strengthening	$\frac{P_{exp}}{P_{the}}$ *		initial	at ultimate	
RCB2	0	0.050	12.70	10	32.82	52	1.58	125.2	10	122	S.F.
PPCB2-1	0.53	0.142	25.23	14	61.96	80	1.30	101.74	10	104	F.F. + L.C. at point load
PPCB2-1c				20		66 no strengthening	1.06	80.62	N/A		S.F.
PPCB2-2	0.70	0.234	37.76	16	87.86	106	1.21	42.15	30	41	cable broke
						96	1.09	74.08		----	beam failed S.F.

Note: N/A: Not Available S.F.: Shear Failure F.F.: Flexural Failure L.C.: Longitudinal Crack

*except for the control beam PPCB1-1c, this ratio indicates the increase in load carrying capacity after strengthening

Table 4.4: Critical Results of Tests on Beams of Group 3 ($S/d_p = 25$) and their Modes of Failure

Beam Designation	*PPR*	ω	Cracking Load (kN)		Ultimate Load (kN)			Deflection at ultimate (mm)	Force in CFCC cable (kN)		Mode of Failure
			Theor.	Exper.	Theoretical without strengthening	Experimental after strengthening	$\frac{P_{exp}}{P_{the}}$ *		initial	at ultimate	
RCB3	0	0.050	9.00	6	22.97	39	1.70	96.56	10	88	F.F. + L.C. at point load
PPCB3-1	0.53	0.142	17.66	20	43.37	58	1.34	154.0	10	78	F.F. + L.C. at point load
PPCB3-1c				14		49 no strengthening	1.13	208.79	N/A		F.F.
PPCB3-2	0.70	0.234	26.43	22	61.50	78	1.27	124.0	30	180	F.F. + L.C. at point load

Note: N/A: Not Available S.F.: Shear Failure F.F.: Flexural Failure L.C.: Longitudinal Crack

*except for the control beam PPCB1-1c, this ratio indicates the increase in load carrying capacity after strengthening

4.9 Stresses in External Prestressing Cables

Knowledge of the stress in the external cables at ultimate is necessary to determine the load-carrying or the strength of an externally prestressed concrete member. Estimation of the stress in the cable at any load level is generally not simple. Since external cables are not bonded to the concrete, the increase in the stress or strain in the cable under applied loads depends on the increase in the cable elongation between the anchorage ends, which in turn, depends on the deformation of the whole member. In other words, the stress change in an unbonded tendon is member-dependent and a sectional analysis based on strain compatibility is not sufficient to provide a complete solution as in the case of bonded tendons.

Generally speaking, the stress in unbonded tendons depend, among other factors, on:

1. The initial effective prestress.
2. The cable profile.
3. The member span to depth ratio.
4. The type and distribution of loading.
5. The amount of prestressed reinforcement.
6. The amount of non-prestressed reinforcement.
7. The material characteristics.
8. Deflected shape of the beam.

In the case of external cables, the following additional factors are to be considered:

1. The possible slip at a deviation point.
2. The change of tendon eccentricity due to the deformation of the structure.
3. Spacing between deviators.

The change of eccentricity of the tendons in between the points of anchorage and deviators with the applied loads and time-dependent deformations leads to "second order effects" which may affect the response of the member and the stress in the prestressing tendons. For a deviated external tendon profile of the type used in this investigation, the increase in tendon elongation and therefore the increase in strain in the tendon is close to being linearly proportional to the increase in beam deflection at midspan (location of saddle or deviator). Virloguex (1988 and 1989) proposed a model for computing the stress at ultimate in internal unbonded cables. He suggested that his model can also be used in the case of externally prestressed cables. Alkhairi (1991) adopted Virloguex's model in his investigation on the behaviour of externally prestressed members. Virloguex's model is explained in the following subsection. A simple equation for the stress in externally prestressed cables has been developed in the present work and presented in Subsection 4.9.2. In subsection 4.9.3, the accuracy of Virloguex's equation and the equation developed in the present work is investigated by comparison of the changes in stresses in the external CFCC cables used in the present work as calculated from the measured deflections with those calculated from the measured forces in the cables.

4.9.1 Virloguex's Model for the stress in Internal Unbonded Tendons at Ultimate

In this model, Virloguex (1988) considered a concrete beam prestressed with internal unbonded cable consist at ultimate of two symmetrical rigid bodies connected together by a hinge at midspan as shown in Figure 4.43. At failure, a mechanism forms and after opening of the central joint, the two rigid bodies are held only by the hinge in the upper part of the beam. The prestressing cable in this mechanism can be assumed piecewise linear as shown and located at midspan at a distance d_p from the hinge. In this mechanism, the central joint opens with an angle of 2θ as shown in the Figure 4.43.

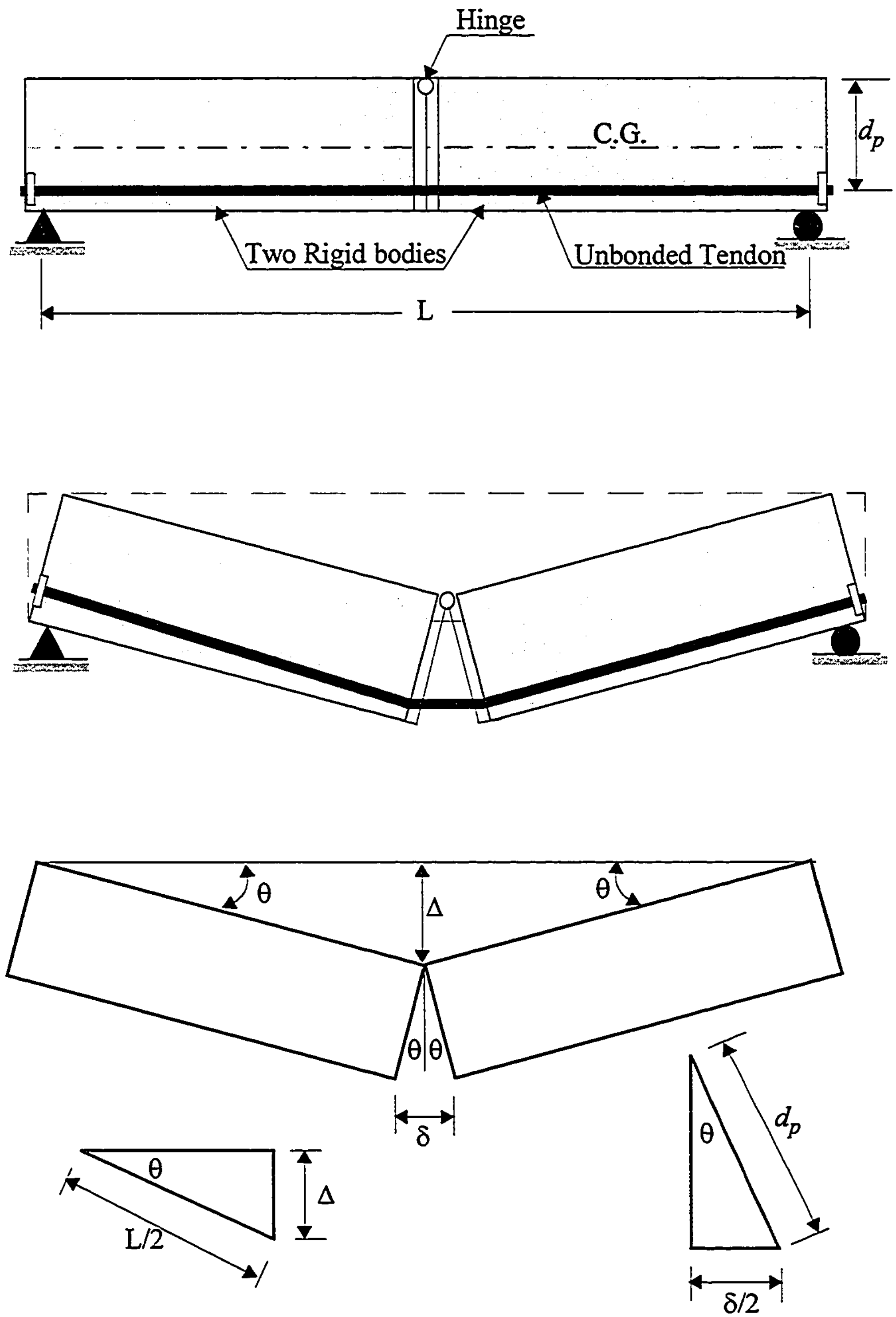


Figure 4.43 Virloguex's Model at Ultimate of Concrete Beam Prestressed with Internal Unbonded Tendons

From geometry of deformations (Figure 4.43), the opening of the joint or the crack can be derived from:

$$\sin\theta = \frac{\delta/2}{d_p} \tag{4.7}$$

and the deflection at midspan can be derived from:

$$\sin\theta = \frac{\Delta}{L/2} \tag{4.8}$$

From these two equations, the crack opening can be written in terms of midspan deflection and depth of prestressing cable as:

$$\delta = \frac{4\Delta\, d_p}{L} \tag{4.9}$$

The elongation of the prestressing tendon depends on the deformations of the whole member length, and thus, the increase in the tendon strain is given by:

$$\Delta\varepsilon_p = \frac{\delta}{L} \tag{4.10}$$

Thus

$$\Delta\varepsilon_p = \frac{4\Delta\, d_p}{L^2} \tag{4.11}$$

At any load level, Virloguex suggest to replace d_p in Equation (4.11) by e_m, the eccentricity of the unbonded tendon at midspan, in order to obtain the increase in tendon strain in terms of the beam deflection at midspan. Thus,

$$\Delta\varepsilon_{pext} = \frac{4e_m\Delta}{L^2} \tag{4.12}$$

As can be seen, this equation is independent of the eccentricity e_s at the anchorage ends. It can be applied for straight horizontal tendons ($e_m = e_s$) assuming a single crack concept and that the two parts of the member to the right and to the left of the crack remain straight during deformation.

The increase in stress in the external prestressing cable is:

$$\Delta f_p = E_p \cdot \Delta\varepsilon_p \tag{4.13}$$

The stress in the cable at any load level is thus given as:

$$f_p = f_{peff} + \Delta f_p \tag{4.14}$$

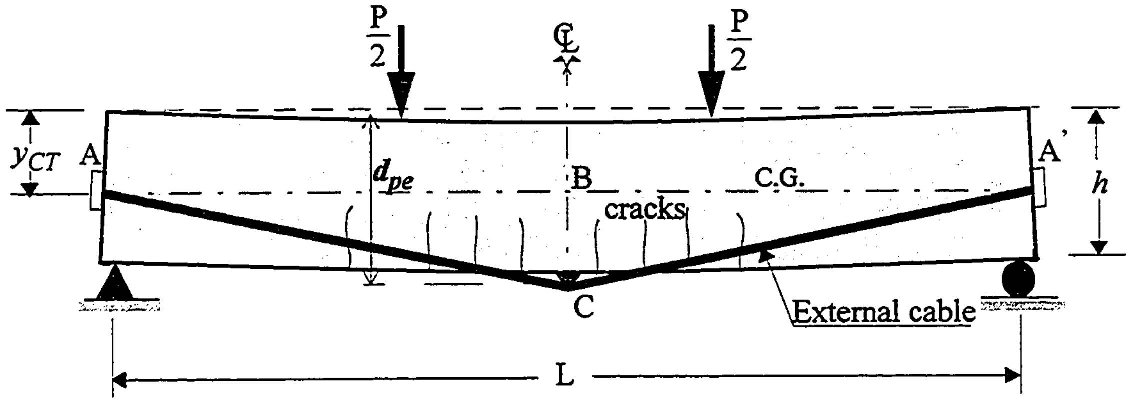
in which f_{peff} is the effective prestressing before loading.

4.9.2 Stress in External Prestressing Tendons

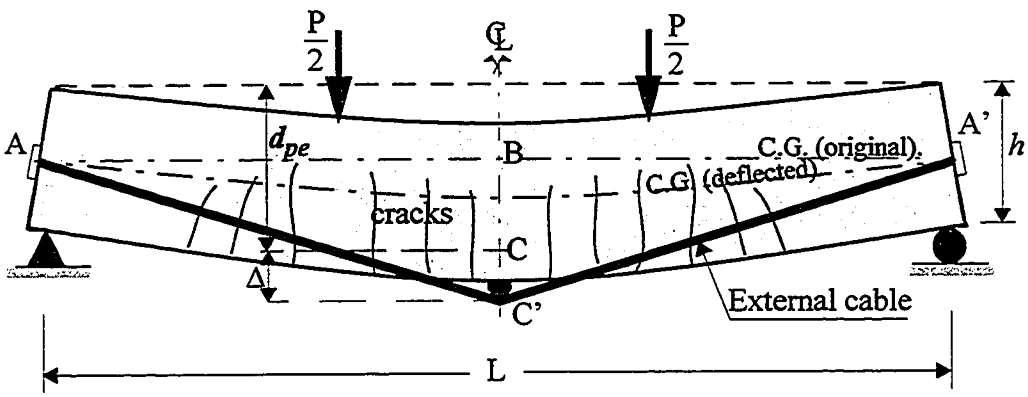
Figure 4.44a represents a concrete member just after being strengthened by means of a single-point draped externally prestressed cable. Points A and A' are the anchorage points assumed coinciding with the centroid of the end sections of the beams (i.e., $e_A = e_A'$ = 0). Thus A and A' are located at a distance y_{CT} from the top fibers of the end sections. Point B is the intersection of the axis AA' with the centerline of the beam. Point C is the centroid of the cable cross section at the saddle point, located at a distance d_{pe} from the top fiber of the concrete section. Thus the distance BC is equal to $d_{pe} - y_{CT}$.

Figure 4.44b represents the deformed shape of the beam under increasing applied load. Point C' represents the level of the prestressing cable after deformation. The distance CC' is the deflection Δ of the beam at midspan. The original cable length between the anchorage and the saddle before increasing the applied load on the beam is (Figure 4.44c).

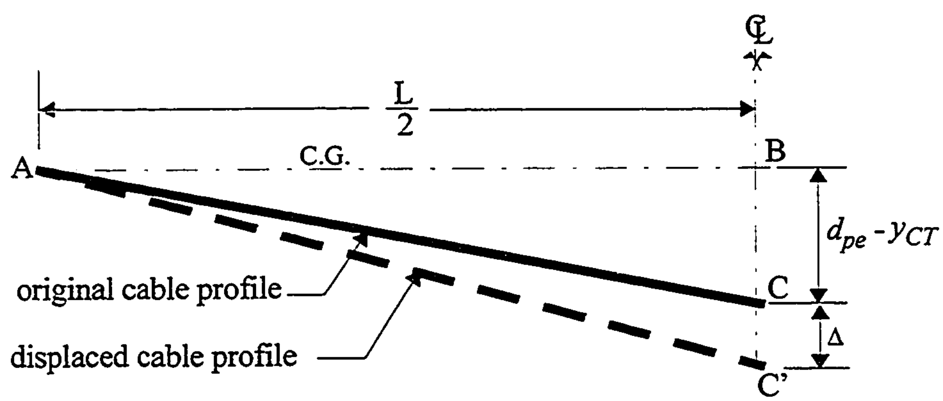
$$AC = \sqrt{(AB)^2 + (BC)^2} \tag{4.15}$$



a) Beam Configuration just after Application of External Prestressing for Strengthening



b) Beam Deformation under Increasing Applied Load



c) Cable Profile before and after Deformation

Figure 4.44 Concrete Member just after being Strengthened

The length of the cable after deformation due to increased applied load is given by:

$$AC' = \sqrt{(AB)^2 + (BC + \Delta)^2} \tag{4.16}$$

The change in length (or the elongation) of the cable in half the span is equal to the difference between the original and the displaced length. Thus:

$$\Delta L = AC' - AC \tag{4.17}$$

and the change in strain in the prestressing cable at any load level is:

$$\Delta\varepsilon_{pext} = \frac{\Delta L}{AC} \tag{4.18}$$

The increase in stress Δf_{pext} in the external prestressing cable can be obtained from:

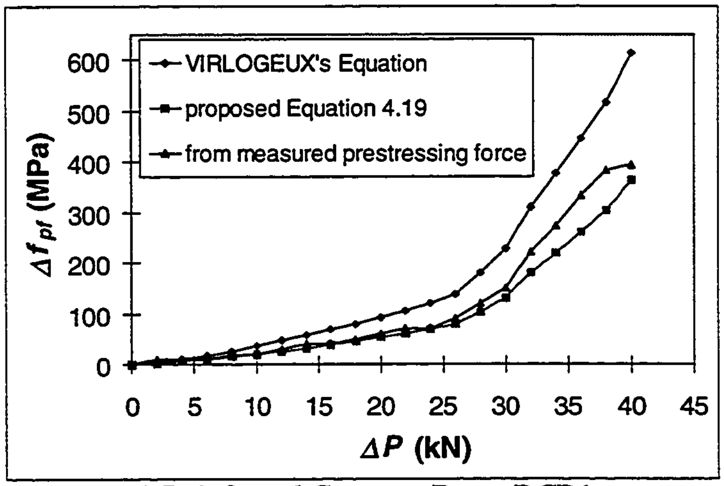
$$\Delta f_{pext} = E_p \cdot \Delta\varepsilon_{pext} \tag{4.19}$$

The change in stress in the external cable can thus be calculated at each load level from the midspan deflection and Equations (4.18 and 4.19). The total stress in the cable can be obtained by adding the change to the initial effective prestress (Equation 4.14).

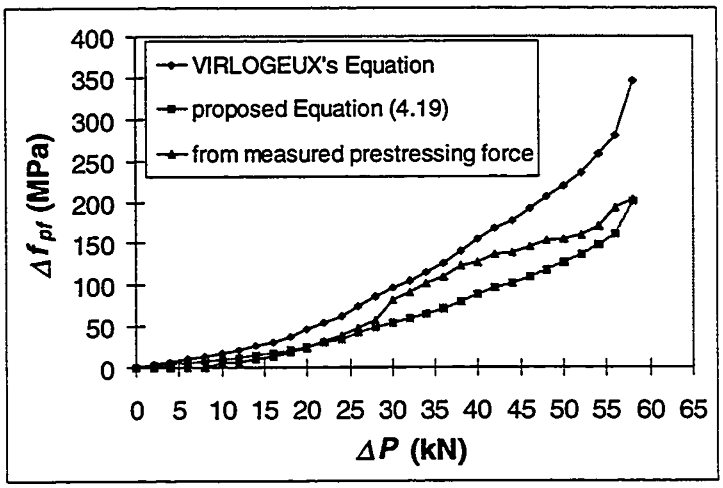
It should be noted here that the axial shortening of the beam was not taken into account when calculating the change in strain of the cable. This axial shortening would be more significant in the case of straight horizontal cable which is not considered here.

4.9.3 Comparison with the Measured External Prestressing Force

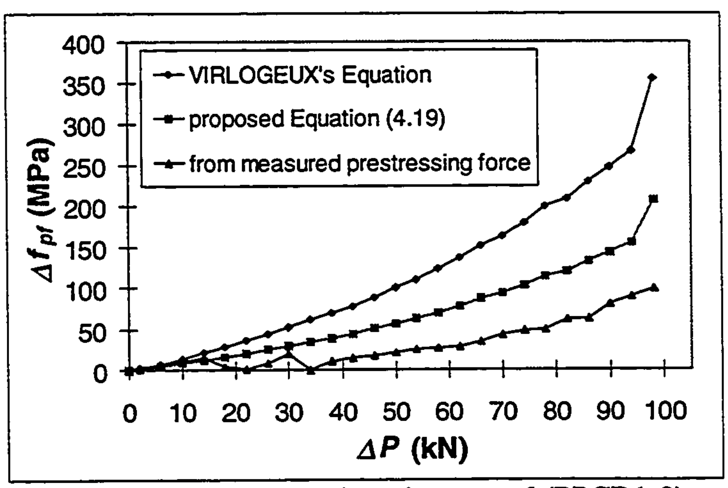
The prestressing force in the external CFCC cables in each of the tested beams was measured using load cells installed at the dead end of the beam (the end where no jacking force was applied). The change of stress in the external prestressing cables was calculated using the measured force at each load increment minus the effective prestressing force introduced at the time of strengthening (the time of jacking) divided by the cross-sectional area of the cable. The values of the stress change thus calculated are plotted in Figures 4. 45 to 4.47 for the nine strengthened beams tested in the present work. The changes in stress calculated by Virloguex's equation (4.13) for unbonded tendons and by Equation 4.19 developed in the present research are also plotted in the same figures 4.45 to 4.47 for comparison with the measured values. The comparison indicate that Virloguex model give higher values for the change of stress in the tendons in all beams. It is more conservative than Equation 4.19. The latter equation, however, gives values of stress change closer to the measured values, particularly at ultimate, for most of the beams.



a) Reinforced Concrete Beam RCB1

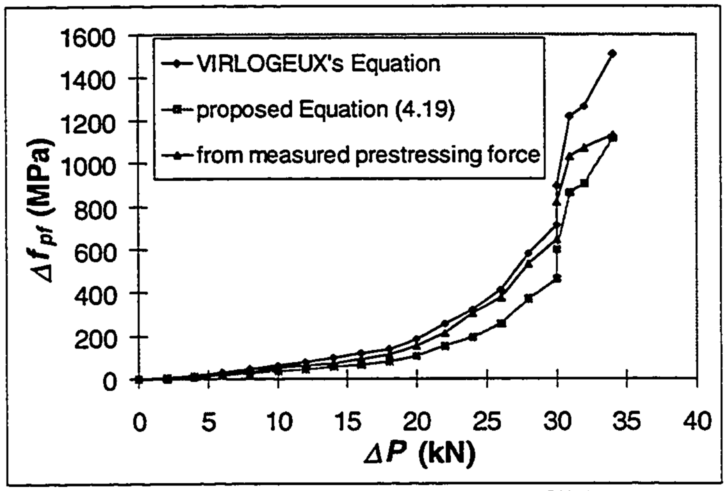


b) PPCB with 1 internal 7-wires strand (PPCB1-1)

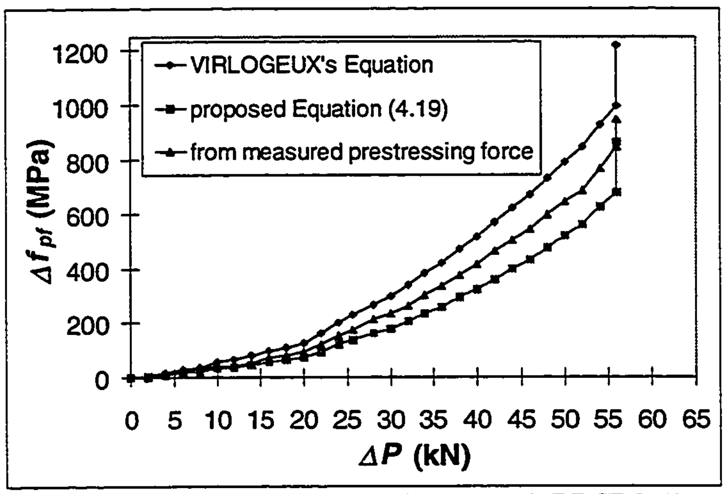


c) PPCB with 2 internal 7-wires strand (PPCB1-2)

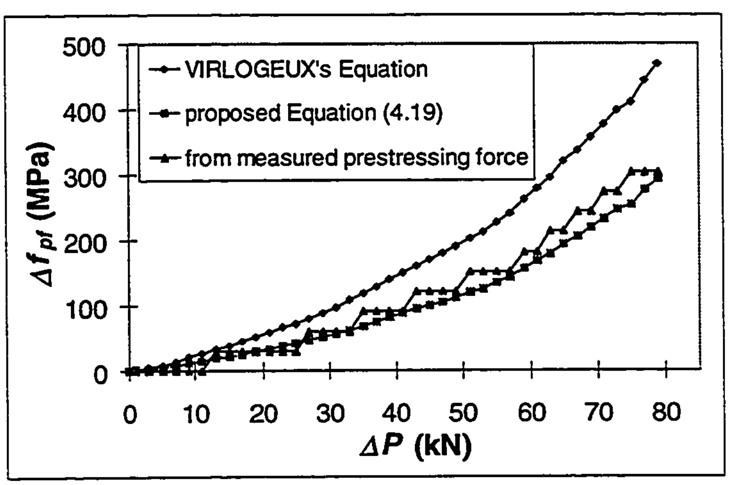
Figure 4.45 Variation of Change in Prestressing in External CFCC Cables with the Increase in Load for Beams Group 1 ($S/d_p = 10.7$)



a) Reinforced Concrete Beam RCB2

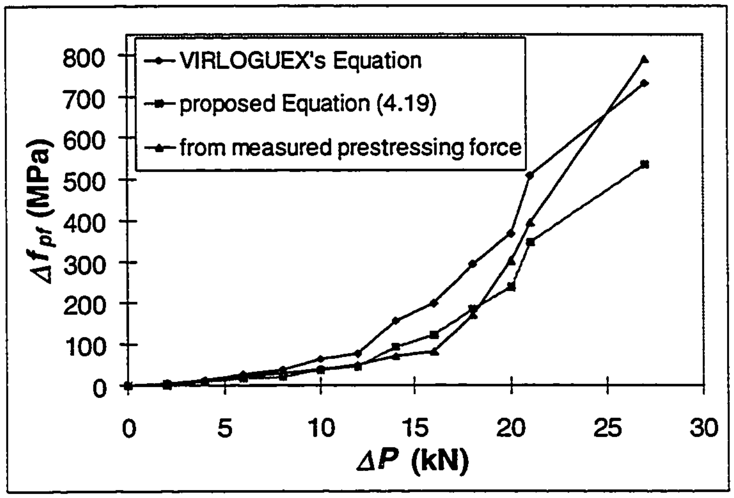


b) PPCB with 1 internal 7-wires strand (PPCB2-1)

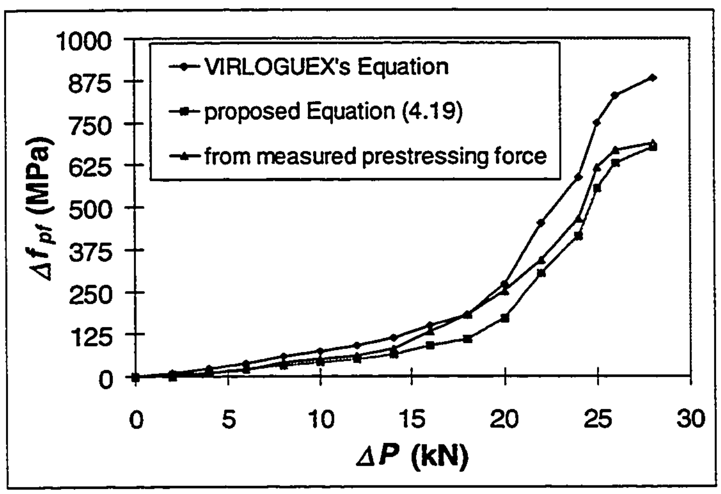


c) PPCB with 2 internal 7-wires strand (PPCB2-2)

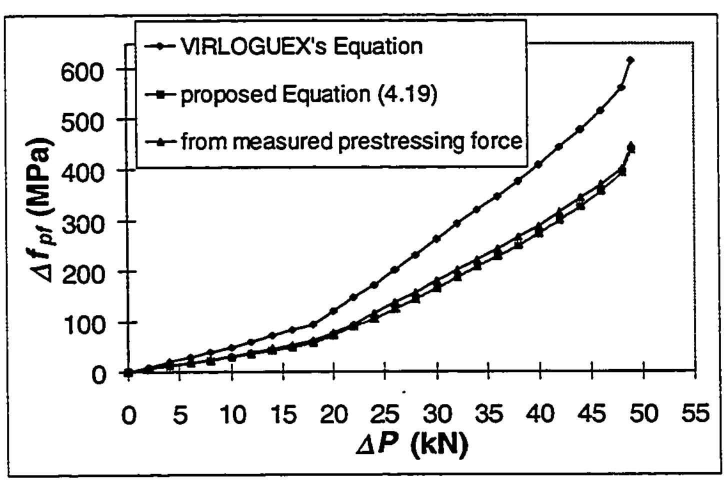
Figure 4.46 Variation of Change in Prestressing in External CFCC Cables with the Increase in Load for Beams Group 2 (S/d_p = 17.8)



a) Reinforced Concrete Beam RCB3



b) PPCB with 1 internal 7-wires strand (PPCB3-1)



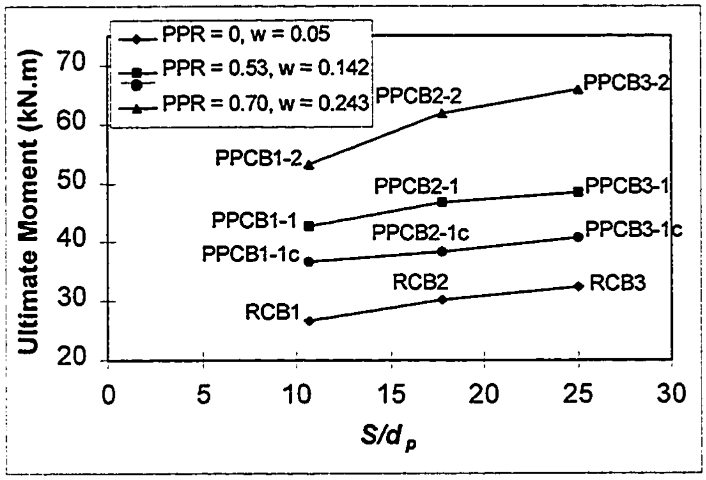
c) PPCB with 2 internal 7-wires strand (PPCB3-2)

Figure 4.47 Variation of Change in Prestressing in External CFCC Cables with the Increase in Load for Beams Group 3 ($S/d_p = 25$)

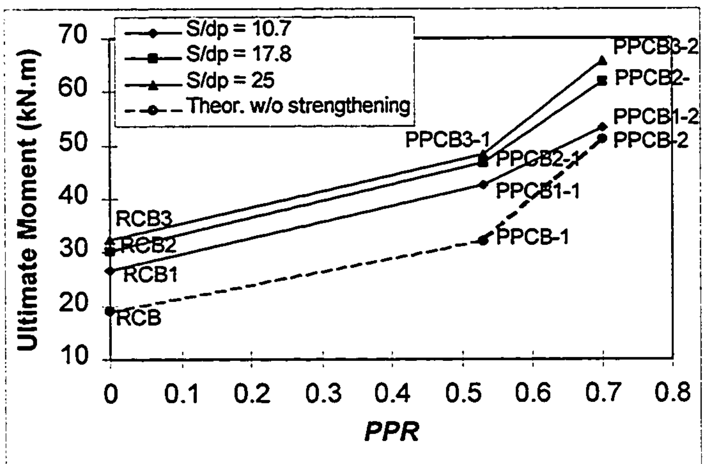
4.10 Effect of Span-to-Depth Ratio (S/d_p), the Partially Prestressing Ratio (*PPR*), and the Reinforcing Index (ω)

Figure 4.48 shows the effects of S/d_p, *PPR* and ω on the ultimate moment capacity of the beams strengthened with the externally prestressed CFCC cables. The measured ultimate moment capacities of the control beams without strengthening are shown in Figure 4.48a for comparison. The figure indicates that increasing S/d_p increases the ultimate moment capacity of the strengthened beams. For the same S/d_p, Figures 4.48b and c indicate that increasing the partial prestressing ratio and the reinforcing index increases the ultimate moment capacity of the strengthened beams. It should be noted that for each partial prestressing ratio, the beams were designed to have equal ultimate moment capacity without strengthening (see Tables 4.2 to 4.4).

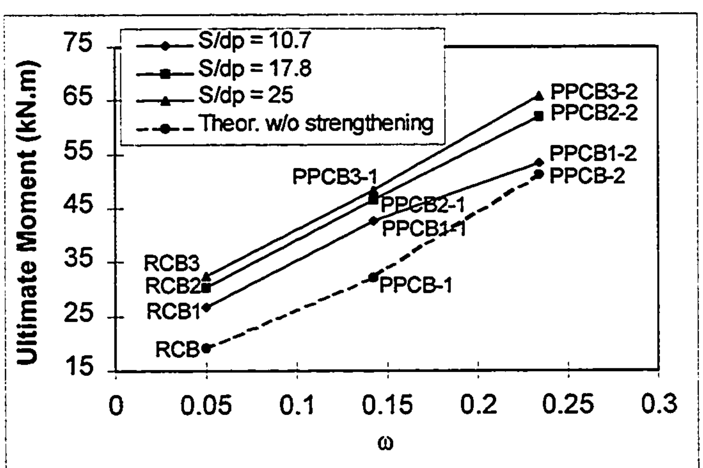
The graphs in Figures 4.49 and 4.50 depict, respectively, the effects of the span-to-depth ratio and the partial prestressing ratio on the change in stress, Δf_{pf}, in the external CFCC cables with the increase in the applied load. The values of Δf_{pf} plotted in the graphs are those calculated from the measured prestressing force in the external cables. As Figure 4.49 indicates, for the same load increment ΔP, the change in stress in the external CFCC cables increases with the increase in the span-to-depth ratio of the beams with equal *PPR* and ω. This can be explained by examination of Equation 4.13 of Virloguex by writing Δ (the deflection) in terms of ΔP and L. The increase in partial prestressing ratio (PPR) or the reinforcing index (ω) increases the beam cross-sectional moment of inertia I, and thus decreases the deflection Δ, and hence, the change in strain and stress in the CFCC cables, according to Equation 4.13 or 4.19 This is confirmed experimentally by the graphs in Figure 4.50 which indicate an increase in Δf_{pf} with the decrease in *PPR* or ω for teach group of beams with the same span-to-depth ratio.



a) Effect of Span-to-Depth Ratio (S/d_p)

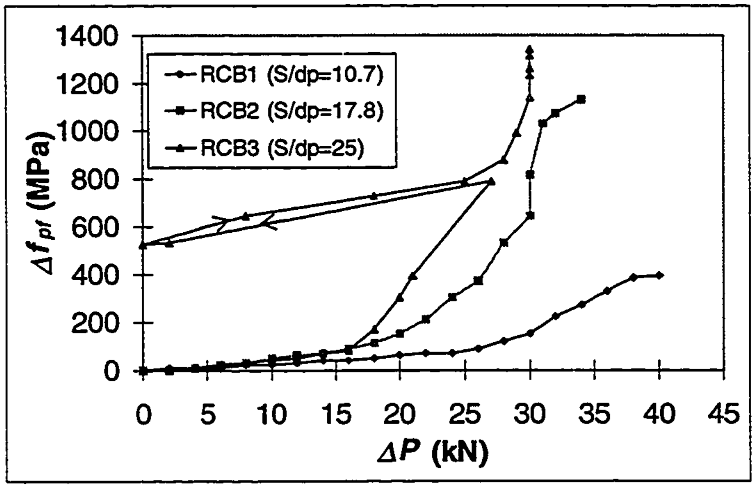


b) Effect of Partial Prestressing Ratio (*PPR*)

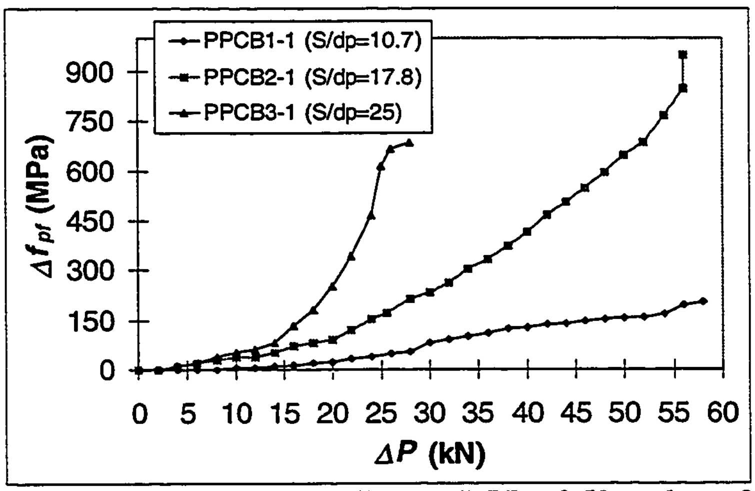


c) Effect of Reinforcing Index (ω)

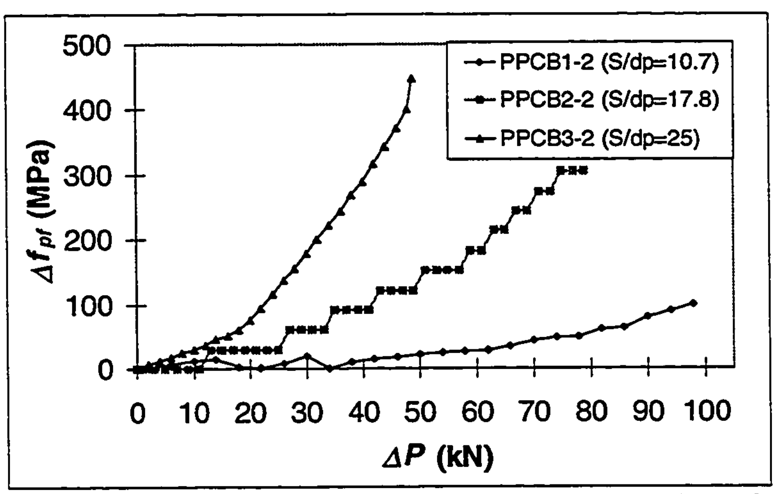
Figure 4.48 Effects of S/d_p, *PPR* and ω on the Ultimate Moment Capacity of the Strengthened Beams



a) Reinforced Concrete Beams ($PPR = 0$, and $\omega = 0.05$)

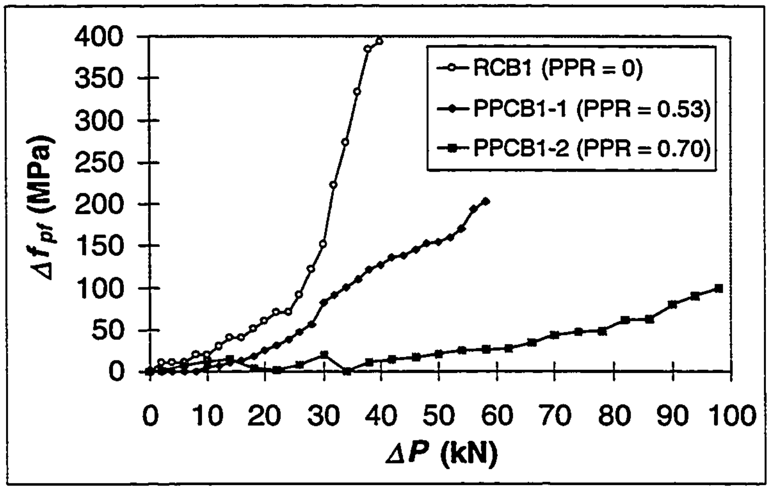


b) Partially Prestressed Concrete Beams ($PPR = 0.53$, and $\omega = 0.142$)

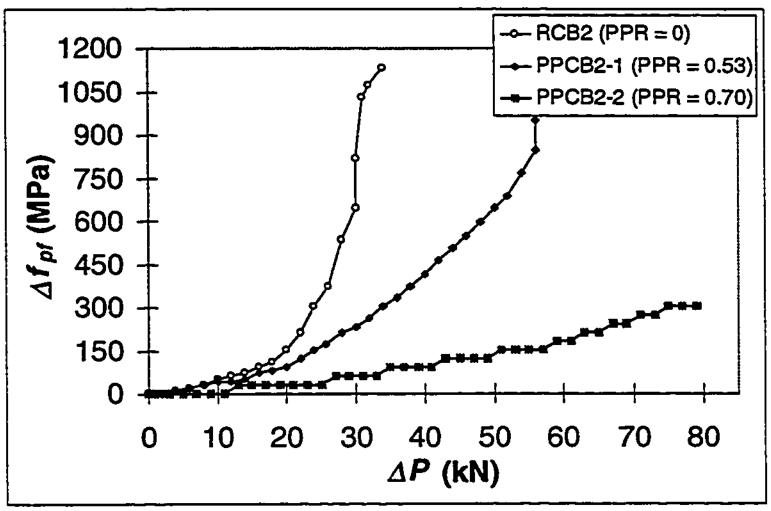


c) Partially Prestressed Concrete Beams ($PPR = 0.70$, and $\omega = 0.243$)

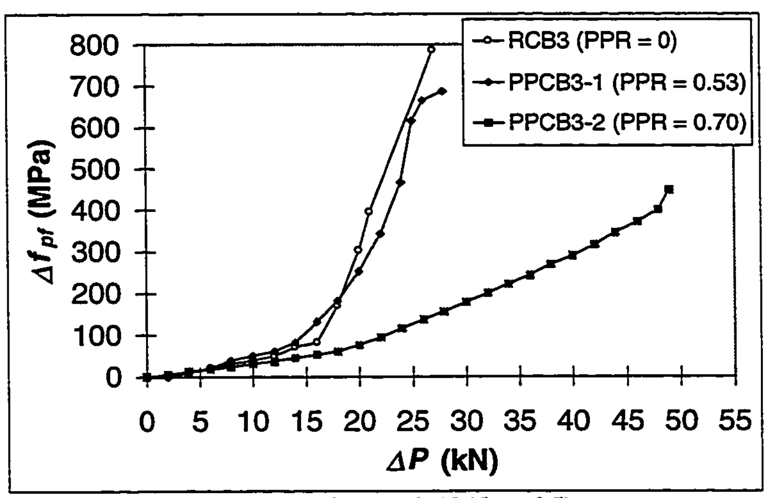
Figure 4.49 Effect of Span-to-Depth Ratio on the Change in Stress in External CFCC Cables



a) Beams Group 1 (S/d_p = 10.7)



b) Beams Group 2 (S/d_p = 17.8)



c) Beams Group 3 (S/d_p = 25)

Figure 4.50 Effect of Partial Prestressing Ratio on the Change in Stress in External CFCC Cables

Chapter 5

Retrofitting and Strengthening of Prestressed Concrete Railway Crossties with Composite Straps

5.1 General

This chapter presents the results of an experimental study on the use of composite to repair and strengthen precast prestressed concrete railway crossties. The research program was decided upon after fine cracks had been observed at the top surface of ties installed on a short span open deck bridge close to Montréal. The purpose of this research was to study the feasibility of retrofitting these ties as opposed to replacing them by new ones. In the experimental program two full scale ties were loaded until a fully stabilized cracking pattern was reached. The ties were then strengthened using composite straps and retested up to failure. The experimental results were compared to those obtained from an extensive investigation conducted at Concordia University on the performance of prestressed crossties without strengthening. The strength of the retrofitted ties was found to be 40% to 50% higher than those tested without straps.

5.2 Background

Most of the North American railway bridges are of the open deck bridge type in which rails are supported on timber ties spanning between two or more main longitudinal bridge girders. Unlike conventional railroad tracks where ties are supported by ballast, such ballast is not incorporated in open deck railway bridge systems. Therefore, this type of bridge has the advantage of reduced dead load which results in smaller bridge girder depth and hence lower cost and greater clearance.

Since the early days of railways, timber ties have been used traditionally all around the world. Over the past two decades the predominant use of timber on railway tracks has been challenged by the advent of prestressed concrete technology. This is mainly because timber crossties can be subjected to deterioration and various types of failure as a result of increase in traffic loads and fluctuations in weather conditions (Rewucki, 1992). Failure of timber ties can be due to splitting, crushing, checking, rotting and mechanical abrasion. Furthermore, the use of close timber crosstie, about 355 mm centre to centre, in open deck bridges and their exposure on all sides make them highly susceptible to fire. Such fire can spread rapidly over the length of the bridge causing considerable damage to the deck and the supporting structures, and results in major traffic disruptions, loss of property and hazard to human life as well as in costly maintenance and repairs. A repair cost of 2.8 million dollars due to fire damage has been reported for one bridge (Kong, 1983). Because of these factors, there has been a significantly high rate of replacement of timber crossties (estimated approximately 17 millions ties per year) over the past few decades.

5.3 Timber versus Concrete Ties

There is a general tendency towards the use of prestressed concrete ties in place of the conventional timber ties in most parts of the world. The reasons for this trend can be summarized as follows:

- The railway traffic frequency, traffic load and train speed have increased significantly over the past 30 years. The increase in traffic load induces unacceptable bending stress in the less rigid timber ties. Also, light traffic loads at high speed produce stress reversals that can cause damage to the rail-timber tie fastening system.
- The growing scarcity of wood of the size and quality, especially hardwood, required for railway bridge ties.

In long straight tracks where long continuously welded rails are more economical, timber tie tracks have had numerous problems due to their misalignment and susceptibility to fire. Such problem can be avoided by the use of more uniform concrete tie track system.

Prestressed concrete crossties on the other hand have the following advantages:

- They have high flexural strength which is suitable for the today's heavy and high speed traffic.
- Their use allows for rigid fastening systems which maintain track gauge and alignment for long time, and leads to fewer irregularities in the track system resulting in smooth ride and low maintenance cost, and overall cost effectiveness over the life cycle of the bridge.
- The estimated life expectancy of prestressed concrete crossties is approximately twice that of timber ties.

Based on the above, and in order to overcome the problems resulting from use of timber crossties, the Canadian National (CN) railways in early seventies, proposed replacing the timber ties with precast prestressed concrete ties placed directly on load carrying bridge girders.

In the CN proposed system, the load is transmitted from the rails to the prestressed concrete ties and to the girder through appropriate bearing pads. The elastomeric pads are placed between the rail and the tie (rail-tie pad), and between the tie and the supporting girder (tie-girder pad). These pads are required since a concrete tie is much harder than a timber tie and may result in a larger impact from wheel loads if the pads were not incorporated. Figure 5.1 shows a typical open deck railway bridge with precast prestressed concrete crossties.

5.4 The Prestressed Concrete Crossties

Because of all the factors described above, the two major Canadian Railways, Canadian National (CN) railway and Canadian Pacific (CP) railway decided to investigate the use of prestressed concrete bridge ties as a viable alternative to the timber bridge ties. The first major commitment towards the use of prestressed concrete ties in Canada was made in 1972 when ties of this type were installed on the CN main line railway track near Jasper, Québec. In 1974, Canadian National (CN) and Canadian Pacific (CP) railways installed their home production of prestressed concrete crossties on most of their tracks.

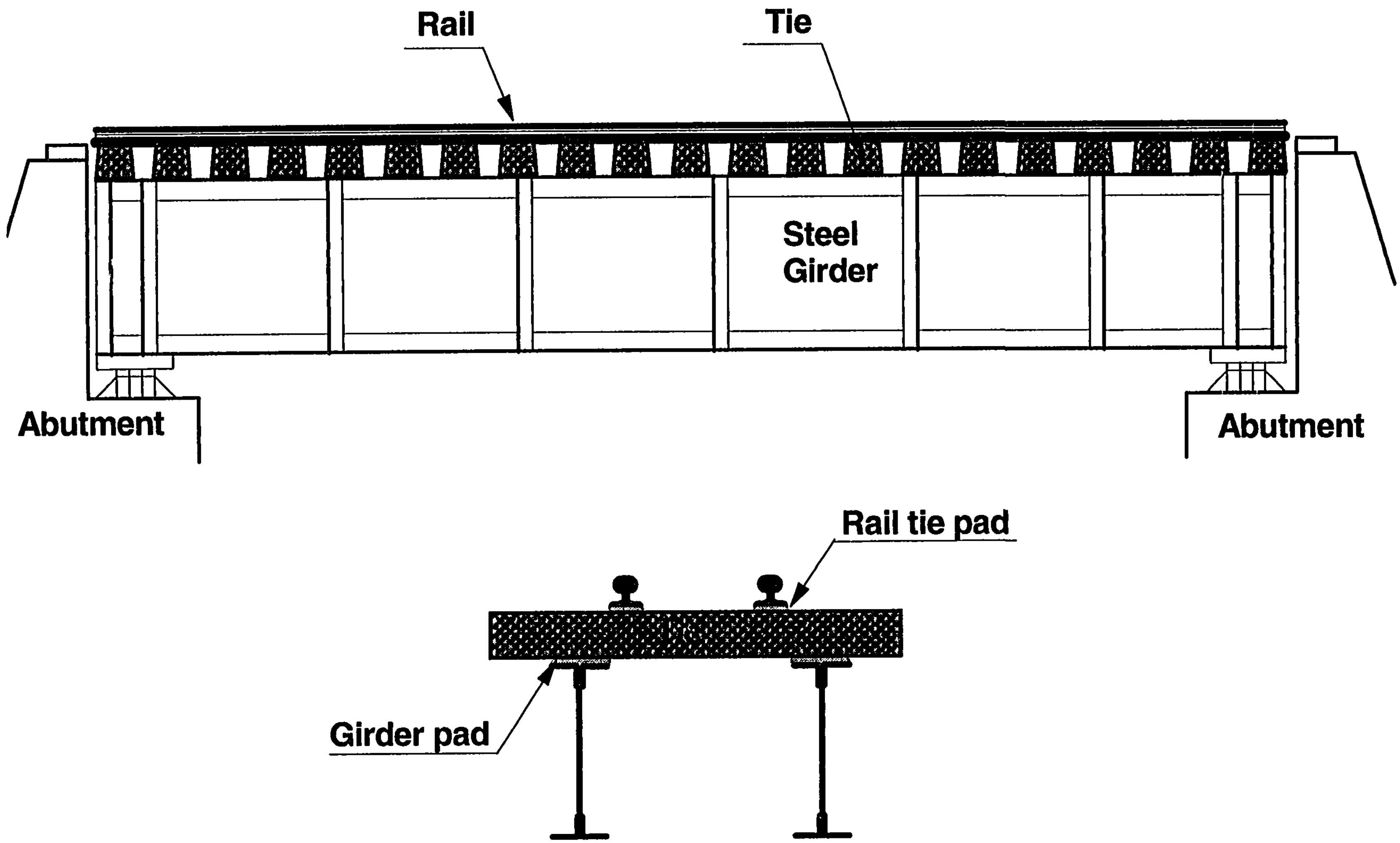


Figure 5.1 Typical Open Deck Railway Bridge.

In September 1984, CN railways installed 22 crossties designed based on a 60% impact factor, on a short span bridge located on its Drummondville subdivision approximately 50 miles (80.5 km) east of Montréal (Scott, 1986a). These ties, shown in Figures 5.2 and 5.3, were 3.66 m long, 305 mm deep, 254 mm wide at the top and 305 mm wide at the bottom. The ties were prestressed with thirty-two 5 mm diameter indented wires to provide a static moment capacity of 72.3 kN.m in positive downward bending and 27 kN.m in negative upward bending moment. The design was based on a bridge girder spacing of 2.44 m, with Cooper E-80 Loading, and one-third of the axle loading plus 60% impact factor assumed to be carried by each tie. Tie spacing was to be 400 mm centre to centre. In 1988, during an inspection, it was reported that 17 of the original 22 crossties had fine cracks at their centers. These cracks started at the top of the bridge crossties and propagated downwards (Peters, 1992). The cracks were attributed to the negative bending moment resulting from the rebound forces on the ties. Presence of such cracks would reduce the fatigue life of the crossties. No cracks due to the positive bending moment were observed.

Similar observations were also reported by CP railways on Bridge No. 24 Winchester Subdivision on their main line in the Smith Falls Division (De Josez, 1987). The bridge is a 33 ft. plate girder consisting of 29 prestressed concrete ties supported by two steel girders installed in place of the usual timber ties in October 1984. Inspection of the bridge concrete ties conducted in October 1986 revealed that a few ties had hairline cracks at the rail seats and centre span at the top fibers. These cracks were caused by vibration of the ties under impact loads which created tension at the top fibers at the rail seat, as opposed to the bottom fibers which remained in compression.

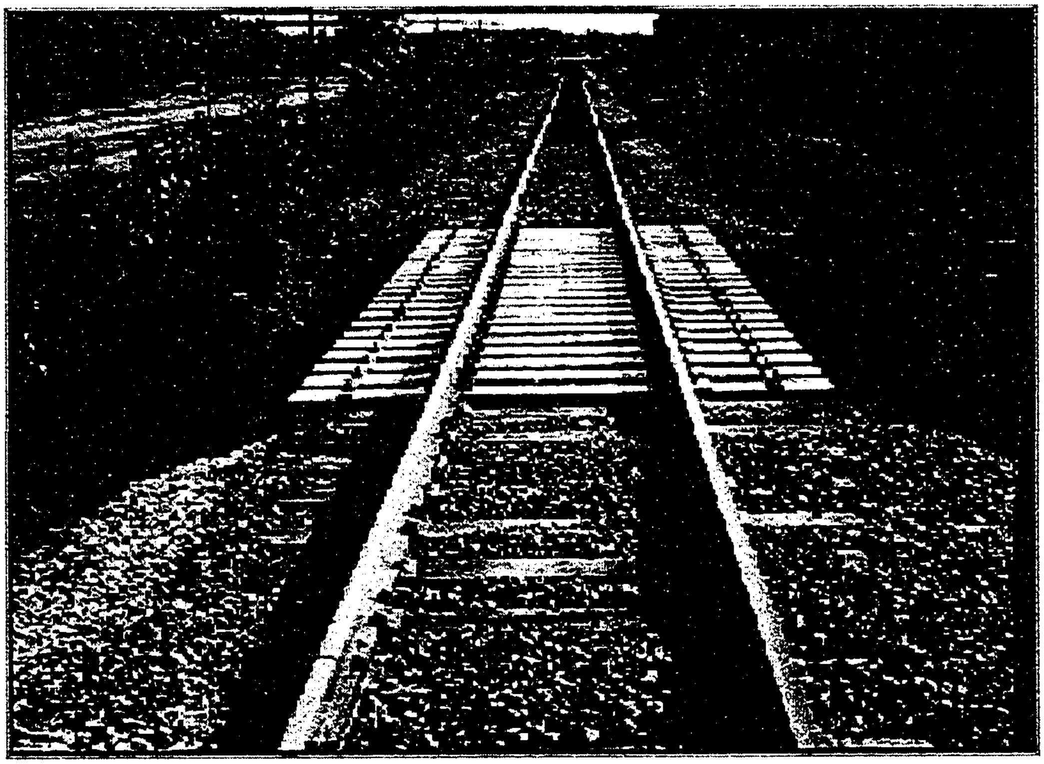


Figure 5.2 The 22 Crossties installed by CN Railways (after Scott, 1986a)

Figure 5.3 Sideview of the 22 Crossties installed by CN Railways (after Scott, 1986a)

Taras 1985 reported that wheel impact loading governs the design of concrete ties. Ties that are subjected to impact undergo resonant bending fluctuation, meaning that they go into reverse bending and suffer far more fatigue loading cycles than those imposed by the simple passing frequency of wheels. Igwemezie and Mirza (1989) conducted an analytical-experimental study to establish the percentage of impact load transmitted to the individual ties. They reported that the maximum tensile stress in the tie occurs at the bottom face underneath the seat of the rail. This is accompanied by a maximum top fiber tensile stress at the adjacent rail seat location, which can result in cracking of the tie.

In an attempt to overcome the problem of occurrence of cracks in concrete railway ties, Mindess and Wan (1995) investigated the impact behaviour of this type of tie. In their investigation, the following parameters were considered either separately or in combination:

(1) adding 0.5% or 1.0% by volume of 30 mm or 50 mm long steel fibers,

(2) adding 5 mm diameter steel stirrups at the rail seat,

(3) reducing f_c' from 60 MPa to 40 MPa, and

(4) changing the level of prestressing.

Their test results showed that:

- Steel fibers greatly improved the tie behaviour, leading to shorter and finer cracks.
- Stirrups were effective when used in conjunction with steel fibers.
- A lower f_c' in conjunction with steel fibers, provided better behaviour than the higher strength but more brittle concrete ties.
- Lower prestressing forces, when combined with addition of fibers appeared to be very effective.

The objective of the current research is to investigate the feasibility of retrofitting damaged prestressed concrete crossties instead of replacing them with new ones. With the invention of advanced composite materials and epoxy adhesives, composite straps have been introduced as a new repair and strengthening technique due to their various advantages. Several investigations have been carried out on the use of epoxy-bonded composite straps for rehabilitation of concrete members. In the following sections, the advantages of this type of material are discussed and the previous research on their use for strengthening of concrete structures is reviewed. The results of an experimental investigation conducted on full scale precast prestressed concrete crossties to examine the efficiency of composite straps made of polyester straps in improving the serviceability and simultaneously enhancing the strength of this type of crosstie are presented.

5.5 Composite Fabrics for Strengthening and Rehabilitation

An established method for upgrading reinforced and prestressed concrete members consists of bonding of steel plates to the tension (cracked) side of the member. Rehabilitation of concrete members using steel plates has been proven to be effective in controlling flexural cracks and deflections, and in increasing the load carrying capacity when the members are subjected to either service or ultimate load conditions. However, the weight of the steel plates makes them difficult to handle and restricts the size of plates that can be used. There are also problems with corrosion of the steel/adhesive interface. The use of this method is limited by the mode of failure of the plated member. Use of thick plates, with width to thickness ratio of less than 50, results in a sudden and brittle failure caused by horizontal cracking. The design is governed by this ratio, as well as by the limiting value of 1.5mm for the glue thickness (McKenna & Erki 1994). Experience from strengthening with steel plates indicates problems related to plate separation due to high local interface bond stresses and peeling forces at the end of plates.

Effective countermeasures against corrosion have been used such as zinc-coating of the steel plates or chromium-nickel alloyed steel. Corrosion can be entirely eliminated if fiber reinforced plastic (FRP) plates or sheets are used because the ionic reactions cannot occur at their surface. Bonded FRP sheets have enormous potential for rehabilitation of deteriorated structures. Flexural strengthening of concrete members by bonded FRP sheets also appears to be feasible. Depending on the type of application, the function of externally bonded reinforcement can be any combination of strengthening, stiffening, crack arrest, or corrosion protection. The flexural strength, stiffness and control of cracking are all enhanced through the application of these sheets, and the resulting behaviour is similar to the behaviour of concrete members strengthened by steel plates.

The composite sheets can be placed with or without prestressing externally on the tension side of the member. The prestressed sheets introduce compressive stresses to the tension face and thus close existing cracks and delay formation of new cracks and increase the stiffness. The process for the field installation of externally bonded FRP reinforcement consists of the following basic steps: concrete surface preparation-cleaning, sealing cracks, rust proofing of existing steel reinforcement, smoothing, etc.; application of a primer coat; application of resin undercoat; adhesion of the sheets; curing; application of the finish coat.

The advantage of using fibre composite materials instead of steel plates is to eliminate the corrosion at the concrete/plate interface which would likely affect the bond strength. The light weight is an important characteristic of fibre composite materials as it leads to easy handling during construction, to no additional load to the structure and to low labor and no scafolding cost. According to Meier and Kaiser (1991), for a specific application in bridge strengthening it is possible that 94 kg of steel plates can be replaced with 4.5 kg of carbon fibre composite sheet. Although CFRP is normally considered an expensive material, on average, for a rehabilitation operation, the cost of the material is approximately 20% of the total cost of the strengthening application, the remaining 80% being labour costs. In general, with regard to the economics of using high priced CFRP materials, it should be noted that the weight/stiffness ratio for carbon fiber is 1/5 of that for steel, and the strength of carbon fiber is considerably greater than common construction-grade steel. Although glass fiber materials are much less expensive than carbon fiber materials, the effective strength of GFRP is less than 2/3 that of CFRP and the effective modulus is about 1/6 of that of CFRP and steel.

5.6 Previous Research

Several investigations have been carried out on the use of epoxy-bonded composite fabrics for rehabilitation of concrete members. In this section a brief review of the previous work done using epoxy-bonded composite materials is presented.

Ehsani and Saadamanesh (1990) conducted tests on beams externally prestressed with 6mm thick GFRP sheet. The test beams measured 200×460×4570 mm. Results of four-point loading showed an increase of the first cracking load in excess of 100 percent over the predicted cracking load of the control beam, and an increase in excess of 400 percent over the predicted failure load of the beam without the plate. Failure of the beams occurred due to the concrete shearing approximately 40 mm above and along the line of the tensile reinforcement. Post-cracking behaviour was linear with hairline cracks along the full length of the beam.

Saadatamanesh and Ehsani (1990a) tested four 150×90×1675 mm beams reinforced with epoxy bonded GFRP sheets 6 mm thick and 75 mm wide for the full length of the test specimens. The only difference between these beams was the type of epoxy used. The authors investigated four types of epoxy. The epoxies selected had a wide range of strengths and ductilities. From the test results the following conclusions were drawn on the type of epoxy used:

- The first epoxy had a rubbery texture forming a bond line, approximately 1.8 mm thick with a required curing time of 24 hours at 25 °C, and a tensile lap shear strength of 13 MPa. It was the most flexible of all epoxies used. The failure was ductile, the bond between the plate and the beam failed gradually and the GFRP plate separation from the beam was due to the high flexibility of the epoxy that allowed for little shear transfer between the plate and the beam. Not too much increase in the ultimate strength was achieved.
- A tougher epoxy with a tensile lap shear strength of 14 MPa was used, and the maximum elongation at failure for the epoxy was 170 percent. The epoxy was cured at room temperature for 12 hours and formed 1 mm thick layer. The results showed that the stiffer epoxy allowed for a larger increase in the overall stiffness of the beam. Failure was attributed to shear cracks resulting in a sudden failure of the beam through plate separation particularly around the base of the large shear cracks where the bond between the plate and concrete had been severally damaged. As result of premature shear failure, the true ultimate flexural capacity was not reached.
- The third type of epoxy was a rubber-toughened epoxy that was relatively viscous and had a consistency similar to cement past. The pot life was approximately 1/2 hour, and

the bond line formed was 1.6 mm thick. The tensile lap shear strength was 14 to 15 MPa according to the manufacturer, and the maximum elongation at failure for the epoxy was 40 percent. The test results showed that an increase in ultimate load of 100 percent over the control beam was reached. There was no visible cracking up to 70 percent of the ultimate load, and thereafter the cracks were very fine and well distributed along the beam. Failure was the result of delamination of a strip of concrete just above the bond line and along the full length of the beam indicating satisfactory performance of the epoxy.

- The fourth epoxy was the most rigid of the four epoxies tested for which the tensile lap shear was not reported but the maximum elongation at failure was only 1 percent. This epoxy resulted in a very brittle and sudden failure which resulted from plate separation due to the formation of flexural cracks. The failure started near the tension cracks and immediately spread throughout the bond line. The plate completely separated from the beam with a shattered sound, and the test was discontinued and no increase in the ultimate capacity was reported.

The authors concluded that the epoxy used in strengthening should have sufficient stiffness and strength to transfer the shear force between the composite plate and the concrete. It should also be tough enough to prevent brittle failure of the bond caused by cracking of concrete in tension. Rubber toughened epoxies are particularly suitable for this application.

Saadatamanesh and Ehsani (1990b) investigated the application of 6 mm thick GFRP sheets to increase the flexural capacity of Tee beams. The beams were 4.6 m long and a cross section composed of 200×460 mm web and a 610×75 mm flange. The results of four-point loading showed that the bonded sheets increased the flexural strength of the beam by 170% higher than the nominal strength. More importantly, they showed that the behaviour of beams strengthened with FRPs is very similar to the behaviour of beams strengthened externally using steel plates.

Meier and Kaiser (1991) conducted tests on 26 flexural beams with a 2.0 m span in which externally carbon fibre composite epoxy resin sheets of 3mm thickness were used for strengthening. The test specimens were 150×200 mm in cross-section. The sheets were 3 mm thick and 200 mm wide. Meier and Kaiser concluded that the ultimate strength of the strengthened beams was almost doubled and the deflection at the ultimate load represented only one half of the unstrengthened beams. It was noted that bonding of a pre-cracked beam with a CFRP sheet resulted in a uniform distribution of cracks along the full length of the member. In comparison with the unstrengthened beams, the sheets led to a much finer crack distribution. However, when subjected to thermal loading (+20 °C to -20 °C) for 100 cycles, there were no negative effects on the load carrying capacity of the strengthened beams. The authors described in details the three main types of failure modes that may appear when FRPs sheets externally bonded to reinforced concrete beams were used. The failure modes observed were described as follows:

- Tensile failure of the CFRP sheets. This type of failure is sudden, but can be easily predicted due to prior extensive cracking in the concrete beam.
- Compression failure of the concrete.
- Peeling-off of the CFRP sheet produced by the relative vertical displacement across shear cracks in the concrete.

Ritchie et al. (1991) tested a series of 16 under-reinforced beams to study the effectiveness of external strengthening using FRP plates. Thin plates of glass, carbon, or aramid fibres were bonded to the tension side of the beams using epoxy resins. An increase in the ultimate strength from 40 to 70% was achieved for the beams with FRP plates compared to that of unplated beams. Final failure generally occurred in the beams at the ends of FRP plates. An analytical method was derived based on strain compatibility to calculate the strength and stiffness of the plated beams.

Fiber reinforced plastic sheets have been already used in strengthening and rehabilitation of existing bridges. Meier et al. (1992, 1993a and b) reported on the rehabilitation of the Ibach Bridge in Switzerland. The bridge suffered accidental damage to a prestressing tendon in the outer web, and as a result, the load carrying capacity of the bridge decreased. It was estimated that 175 kg (386 lb) of steel plates would have been required to strengthen the bridge to its original design load. However, only 6.2 kg (13.7 lb) of carbon fiber reinforced plastic (CFRP) sheets were used instead of the steel plates, resulting in a 28.2:1 reduction in weight. Moreover, the repairs were quickly made at night with a lightweight mobile man-lift platform rather than with more heavily rated (and more expensive) lifting equipment.

They also conducted tests on 26 Tee-beams externally reinforced with CFRP sheets epoxy bonded to the tensile face of the web. The beams were 6 m long with a cross section composed of 260×340 mm web and a 900×160 mm flange. The beams were externally reinforced with 1 mm thick by 200 mm wide CFRP sheet bonded to the tensile face of the web by epoxy. The effects of both non-pretensioned and pretensioned CFRP sheets were investigated. The experimental results showed an increase of 32% in the maximum load for both pretensioned and non-pretensioned T-beams. Only one difference between the two methods was observed. The deflection at failure of the non-pretensioned beams was the same as the control beam, while the deflection of the pretensioned beam, at maximum load decreased by approximately 100%. From the test results, they concluded that the application of FRP sheets had to be performed very carefully because peeling-off of the sheets might lead to premature failure of the beam. Also, the prestressing of the sheets was mentioned as a significant contribution towards the improvement of serviceability of structures. In addition to the three types of failure modes described originally in 1991, the authors observed the failure modes shown in Figure 5.4 with corresponding numbered labels (Meier at al. 1992):

(1) Tensile failure of the CFRP sheet: the sheets failed more or less suddenly, with a sharp explosive snap. The impending failure was always announced far in advance by cracking sounds.

(2) Classical concrete failure in th compression zone of the beam.

(3) Continuous peeling-off of the CFRP sheets due to an uneven concrete surface . For thin sheets (less than 1 mm) applied with a vacuum bag, an extremely even bonding surface is required. If the surface is too uneven, the sheet will slowly peel off during loading.

(3) Shearing of the concrete in the tensile zone (also observed as secondary failure).

(4) Interlaminar shear within the CFRP sheet (observed as secondary failure).

(5) Failure of the reinforcing steel in the tensile zone. This failure mode was only observed during fatigue tests.

(6) Cohesive failure within the adhesive .

(7) Adhesive failure at the interface CFRP sheet/adhesive .

(8) Adhesive failure at the interface CFRP concrete/adhesive .

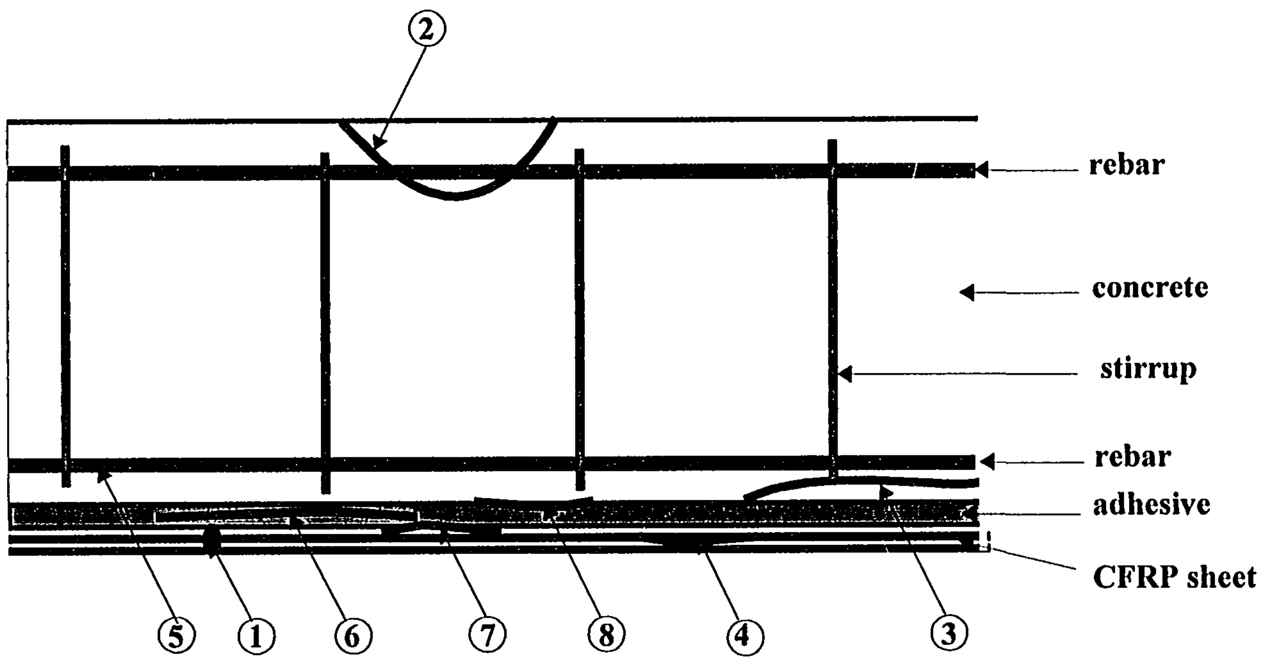


Figure 5.4 Failure Modes (after Meier et al. 1992).

The authors mentioned that the last three failure modes described above were not observed during the load tests but they are theoretically possible. Also, they recommended that for post-strengthening with CFRP sheets that the CFRP sheets should fail during yielding of the reinforcing rebars before failure of the concrete in the compression zone occurs.

Deblois and Beaulieu (1992) investigated the use of GFRP sheets to externally reinforced concrete beams. The sheets were applied either as unidirectional sheets with thickness of 4.75 mm or as bidirectional sheets with thickness of 6.35 mm. Two method of bonding were used: epoxy adhesive or a combination of epoxy and bolts. The experimental results showed that when the reinforcing sheets were used as bidirectional, the ultimate load increased by 32%. The attachment of sheets resulted in an increase of 34% when a combination of epoxy and bolted bonding was used. Only 18% increase in the ultimate load was obtained using epoxy bonded unidirectional sheets. It was noted that for all tests the application of GFRP sheets decreased the ductility of the beam and reduced deflection at maximum load. They reported also that the combination of bolted and bonded anchorage system may improve the static and fatigue response. The authors developed an analytical model to predict the behaviour of the externally bonded reinforced beam. The analytical model predicted well the failure of externally reinforced beams and the types of failures observed were similar to those reported by Meier et al. (1992).

Triantafillou, Deskovic, and Deuring (1992) investigated the strengthening of flexural members using prestressed CFRP sheets. Unidirectional sheets were first pretensioned and then bonded to the tension face of the beam. Once the epoxy adhesive was fully cured, both ends were cut, and the sheet and the beam were transformed into a prestressed element. Five beams were tested under three-point loading. The results of the flexural tests showed an increase of 3 to 4 times in the ultimate load over the control beam. Failure of all prestressed beams consisted of diagonal cracks, which caused localized peeling-off of the sheets and subsequent slipping. All of the beams experienced an increase in the load carrying capacity until the concrete in the compression zone started to crush and the sheet debonded. The authors reported that FRP prestressed concrete members exhibited excellent strength, stiffness, and ductility characteristics provided that the external reinforcement is maintained along all the length of the member and over the supports. The authors developed an analytical model to account for the maximum achievable prestress of external FRP sheets, and verified it with test results.

Triantafillou and Pelvris, (1992) reported that FRP-strengthened concrete beams can fail in several modes when loaded in bending, and identified the collapse mechanisms as follows:

1. Steel yield - FRP rupture: when the FRP area fractions of the section are small.
2. Steel yield - concrete crushing: when the FRP area fractions of the section are high.
3. Compression failure: when the FRP area fractions of the section are high and steel have not yielded.
4. Debonding which may occur due to:
   a. Sudden propagation of cracks in the adhesive (most resin adhesive are brittle)
   b. Peeling-off of the FRP sheet because of shear crack in concrete.
   c. Shear failure of the concrete layer between the FRP sheet and the longitudinal steel reinforcement

McKenna and Erkie (1993) investigated the flexural behaviour of concrete beams, with and without external strengthening using carbon fiber and glass fiber sheets bonded to their tension faces. The experimental program contained sixteen 150×300×2000 mm reinforced concrete beams, and thirty 150×150×1000 mm plain concrete beams. The parameters considered in the investigation were the influence of the concrete strength on the behaviour of the strengthened beams, the failure mechanisms of the strengthened beams, and the differences in behaviour between beams strengthened with carbon fiber sheets and glass fiber sheets. Different number of sheets and fiber orientation were used in the experimental program. The effect of number of layers and varying fiber orientation was significant on the ultimate load and the stiffness in the case of reinforced concrete. The experimental results showed that the compressive and tensile strength of the concrete does not affect the FRP failure load. Failure was due to rupture of the sheets in the constant moment region and sheet delamination as a result of relative displacement at the flexural-shear crack. Also, shearing of the concrete at the epoxy resin interface was observed. The load versus displacement behaviour of the carbon fiber strengthened concrete beams was repeatable compared to that of the glass fiber members.

Drimoussis and Cheng (1994) investigated the feasibility of strengthening existing concrete bridge girders removed from a demolished bridge for shear deficiencies using carbon fiber reinforced plastic CFRP sheets bonded to the webs. They found that shear failure was governed by the strength of the concrete rather than the CFRP material. Also failure of the sheets was generally by peeling off. This refers to the sheet peeling the concrete away such that the failure is in the concrete rather than in the bond line. Also the results showed an increase in the shear capacity of up to 73%. The authors concluded that using externally CFRP sheets to strengthen the girders improved their shear capacity significantly, and suggested that this method could be readily applicable in the field. The tests indicated however that particular attention must be paid to anchorage of the sheets. The best results and behaviour obtained were when the sheets were extended up onto the underside of the top flange and down around the bottom flange.

Blais and Picard (1994) developed a theoretical model to study the effects of various parameters on the flexural behaviour of strengthened beams with composite plates. To verify the theoretical results, they conducted an experimental program on twelve reinforced concrete rectangular beams strengthened with glass fiber composite plates glued and/or mechanically anchored on the tension surface of the beams. The experimental results showed a significant increase in the flexural strength varying between 35% and 95% for the beams strengthened with composite plates depending on the strengthening plate thickness and width. All failures were due to crushing of concrete in the compression zone, in the constant moment region of the beams. According to their test results, the ideal behaviour would be obtained with glued and anchored strengthening plates.

Char, Saadatmanesh, and Ehsani (1994) conducted a parametric study on the flexural strength of concrete girders externally prestressed with epoxy bonded glass fibre reinforced plastic (GFRP) plates, and demonstrated the effectiveness of this strengthening technique for upgrading the load carrying capacity of existing bridge girders. They found that external prestressing of concrete girders with epoxy-bonded plates significantly increase the ultimate load capacity.

Steiner (1996) demonstrated the outstanding advantage of strengthening of reinforced concrete structures with CFRP strips. The CFRP strips used had a high tensile strength in the direction of the fibers. Steiner also observed that in case of transverse loads, longitudinal cracks might occur, which however would not influence the longitudinal tensile strength of the reinforcement. Interlaminary shear strength could be important with respect to transmission of the loads. He argued that, in most cases this is not relevant because the strength of the concrete is the critical factor. He showed that the light weight of the CFRP strips, flexibility and the stability of the epoxy adhesive allow to install the strips without any lifting, supporting and clamping equipment. Also from the aesthetic viewpoint, it is possible to cover the CFRP strips with a thin layer of mortar, plaster or wood. Comparison between CFRP strips and steel plates in terms of strengthening was as described in Table 5.1

Swamy, Lynsdale, and Mukhopadhaya (1996) reported from experimental results that it is possible to obtain ductile failure from a GFRP plate bonded to reinforced concrete beam. Five beams strengthened by GFRP plates were presented having rectangular cross section 150×250 mm with a span of 2800 mm. The plates used 150×3.5×2700 mm and had ultimate tensile strength of 309 MPa and tensile modulus of elasticity of 22 GPa. The adhesive thickness used for all tests was 1.5mm, with a compression strength of 75 MPa, tensile strength of 24 MPa and a 3 day compressive strength development of 70 MPa as supplied by the manufacturer. The specimens were tested in two-point loading positioned at one third span length.

Table 5.1: Comparison between CFRP strips and steel plates (Steiner 1996)

Criteria	Strengthening with	
	CFRP strip	Steel plates
Own Weight	Low	High
Tensile strength	Very high	High
Overall thickness	Very low	Low
Corrosion	None	Yes
Length of plate	Any	Limited
Handling	Easy, flexible	Difficult, rigid
Load bearing	Longitudinal direction only	In any direction
Lap joints	Easy	Complex
Fatigue behavior	Outstanding	Adequate
Materials costs	High	Low
Installation costs	Low	High
Application	No equipment	with lifting equipment and clamping devices

Hutchinson, and Rahim (1996) described the influence of various parameters on the flexural behaviour of beams strengthened with externally bonded FRP reinforcement. These parameters include the concrete beam strength and the modulus and strength of the FRP external reinforcement. The beams were externally strengthened with epoxy-bonded unstressed CFRP and GFRP unidirectional laminates. The experimental results showed that the glass laminates possessed a lower modulus and strength per unit thickness than the carbon laminates. GFRP provided significant ductility and reasonable strength, whilst increased beam enhancement was generaly achieved at the expense of a loss in ductility with increasing thickness of CFRP.

Previuos research also indicates that the selection of suitable adhesive epoxy is very important for the use in strengthening techniques. It should have sufficient stiffness and strength to transfer the shear force between the composite materials and the concrete, and should be tough to prevent brittle bond failure as a result of cracking of concrete (Saadatamanesh and Ehsani 1990a). The ultimate capacity of concrete members increases when the epoxy performs well. Some points should be taken into consideration when bonding steel or plastic sheets to concrete members; these can be summarized as follows: increasing the adhesive thickness reduces the bond stress, the governing failure mode for thick adherends is peeling. In addition, there are two modes of failure of bonded connections: adhesion failure and cohesion failure. Adhesion failure occurs at the interface when the adherend and the adhesive break apart, cohesion failure occurs within the adhesive itself as a result of crack propagation.

5.7 Experimental Program

5.7.1 Tie Specimens

Tests were conducted on two full-scale crossties with different levels of prestressing and different arrangement of pretensioned strands in the compression zone of the tie. The crossties, denoted below as T1 and T2, are of the same size as the ties used by CN Railways in the bridge east of Montréal. Each tie is 3.66 m long with a trapezoidal cross section, 305 mm deep, 254 mm wide at the top and 305 mm wide at the bottom. The cross section properties and strand arrangement of crossties T1 and T2 are shown in Figure 5.5.

5.7.2 Concrete Strength

The specified 28 day concrete compressive strength of the ties was 41 MPa.

5.7.3 Prestressing Steel

Ties T1 and T2 have a total of 14 and 16 prestressing strands, respectively. The increase in number of strands in T2 was intended to provide adequate precompression at the top fibres to resist the stress reversals due to wheel impact. The effective prestressing forces were 700 kN and 840 kN for T1 and T2, respectively. High tensile seven-wire strands of nominal diameter 9.5 mm and nominal cross-sectional area of 55 mm^2 with an ultimate tensile strength of 1860 MPa were used for prestressing.

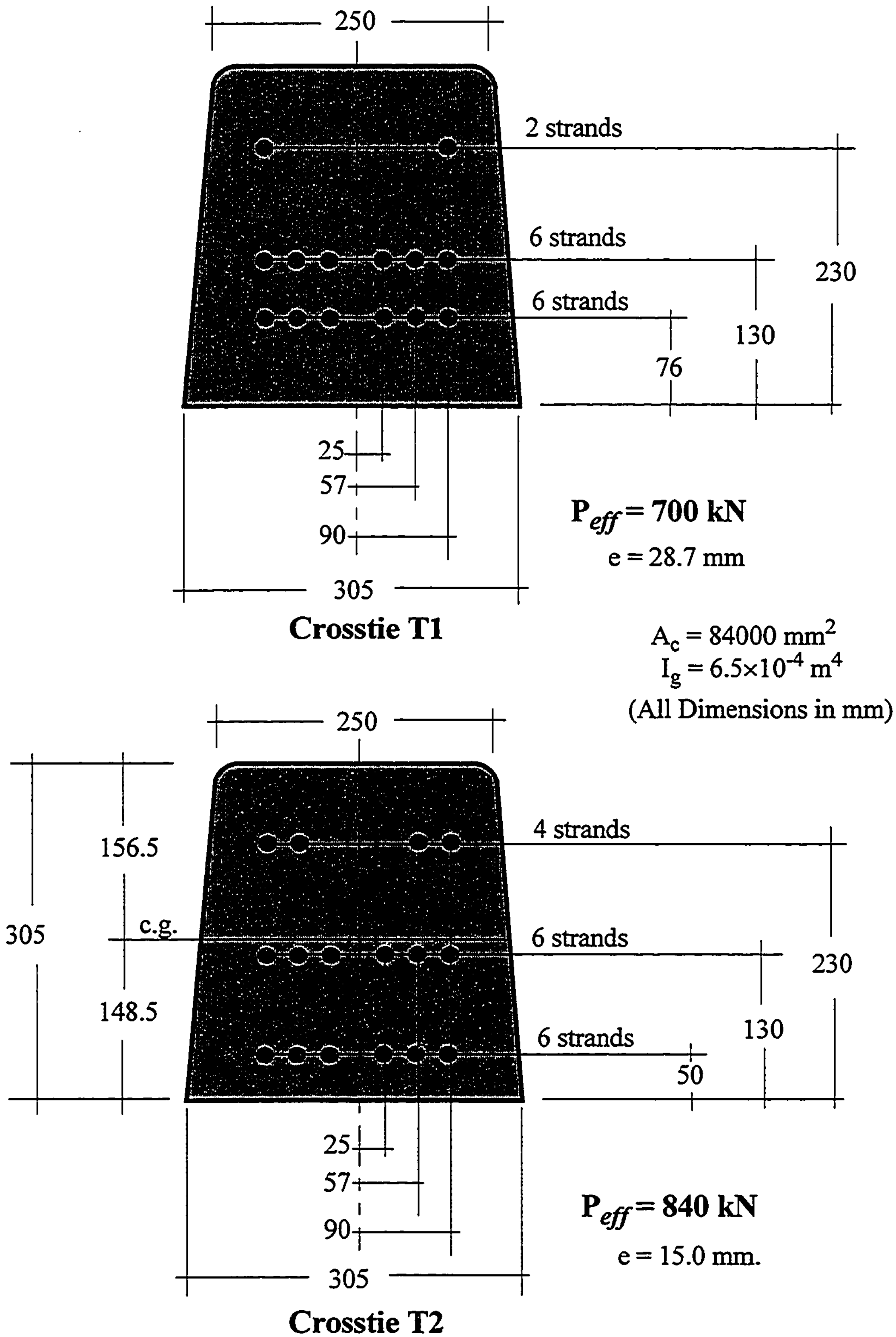


Figure 5.5 Cross Section Properties and Strand Arrangement of Crossties T1 and T2

5.7.4 Loading System

The ties were supported on two steel chairs spanning 2.44 m centre to centre representing the spacing between the open deck bridge girders. Two profile rails at standard railway gauge 1.524 m were placed on the top surface of the tie. Two concentrated loads 1.524 m apart, were applied from the bottom upward to simulate the rebound force from the rail vehicles. Two hydraulic jacks of 500 kN capacity were used for applying the loads. Figure 5.6 shows the test-setup of one of the crossties. The crossties were loaded on their entire bottom width by using hard plate. The same type of plate was used to distribute the reaction forces at the supports. The lateral movement of the ties was not prevented and the ties were allowed to rotate under the applied load.

5.7.5 Instrumentations

The ties were instrumented to measure the applied load, deflections at midspan and at the loading points, and strains at the extreme compression and tension fibres of the crossties. Concrete strains were measured using 120 Ω electrical resistance linear strain gauges of type N11-FA-60-120-11 and 60 mm length manufactured by Showa Measuring Instruments Co. Ltd. installed at midspan on the top and bottom faces of the ties to monitor the tension and compression strains. Mechanical dial gauges with travel sensitivity of 0.01 mm were used to measure the deflection of the ties at five locations shown in Figure 5.6.

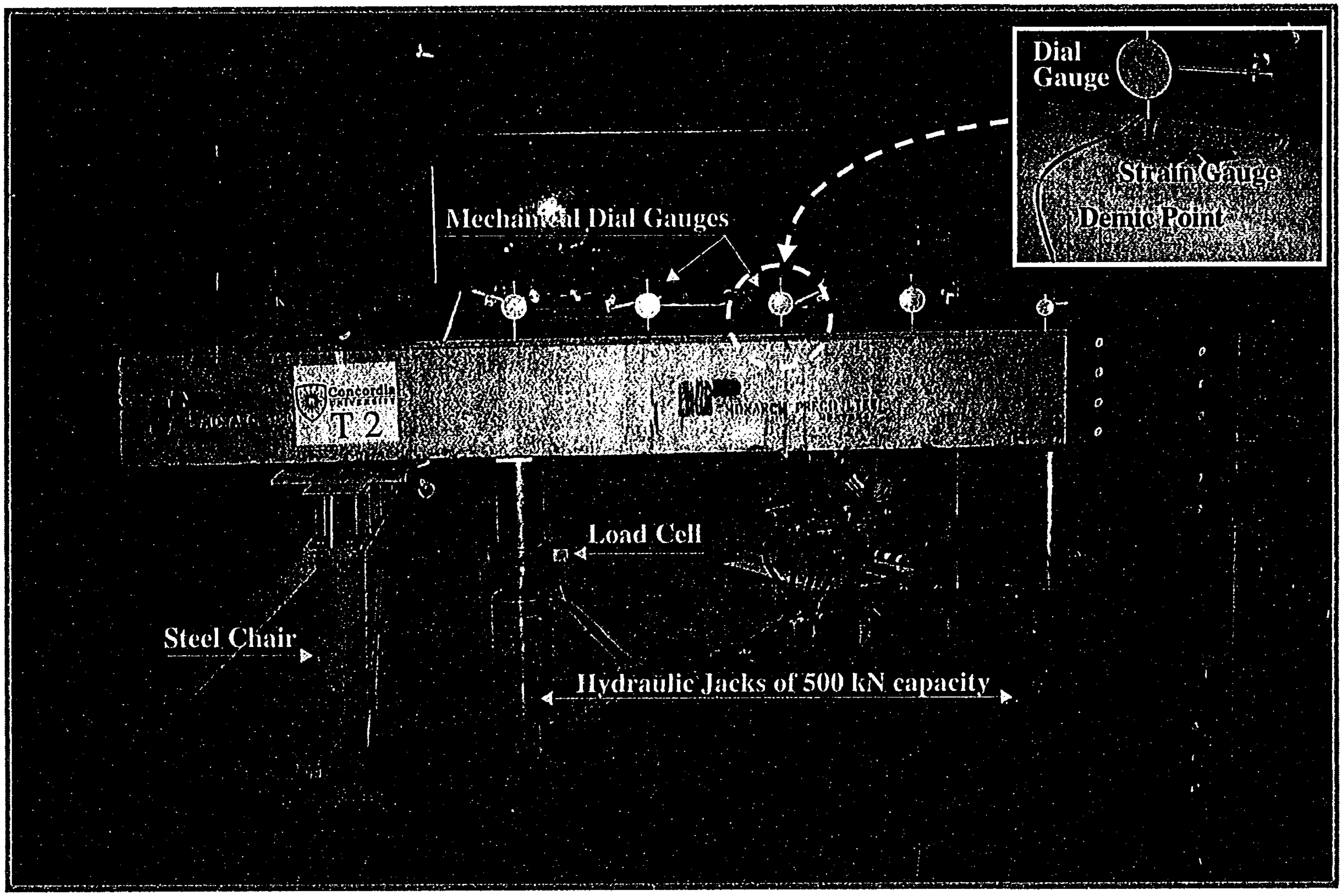


Figure 5.6 Test Set-up of One of the Crossties

5.7.6 Testing Procedure

The load was applied gradually on the crossties until the cracking pattern in the constant moment region was fully developed and reached the stabilized state. At the end of each load increment, the crack propagation was marked and measurements of midspan deflection, strain gauge readings and crack opening were recorded. In order to make the observation of crack propagation conveniently, the cracks were outlined with a black marker. The load was increased up to 1.55 times the load at first cracking; the stage at which the deepest crack propagated to almost two thirds the depth of the crossties. The applied load was then removed and the crossties were strengthened using composite straps placed on the top surface of the tie before re-testing up to failure.

5.7.7 Cracking and Ultimate Loads

The theoretical and experimental cracking and ultimate loads sustained by the crossties without straps are given in Table 5.2.

As illustrated in Table 5.2, the theoretical cracking moment gives an estimate of the cracking moment which is most dependent on the stress state of the concrete, however, the cement mix, strength and size of aggregate and water cement ratio also have an influence. In order to check the ultimate moment capacity of the ties, the strain compatibility method has been used. This method more accurately predicts the ultimate moment capacity of the tie. This was useful in determining the required strength of the load frame and capacity of the jacks for laboratory testing. It should be noted that no reduction capacity factor was used in the theoretical ultimate moment capacity calculations.

Table 5.2: Cracking and Ultimate Capacity for Ties T1 and T2.

Tie type	Theoretical cracking load	Experimental cracking load	Theoretical ultimate load without straps	Experimental ultimate load without straps	$\frac{P_{u_{expe}}}{P_{u_{theo}}}$
T1	67 kN	80 kN	160 kN	158 kN	0.98
T2	98 kN	100 kN	193 kN	172 kN	0.89

5.7.8 Composite Straps

Several procedures are recommended for crack repair. In this study the technique of using composite straps was adopted to strengthening the crossties and repair the cracks. Epoxy-bonding fiber composite straps are used to improve the strength and serviceability of concrete girders. The advantage of using fiber composite straps instead of steel plates is to eliminate the corrosion at the concrete/plate interface which would adversely effect the bond strength.

The full scale crossties tested in this study were strengthened using 105 WGSD Polyester straps supplied by Caristrap International Inc. The synthetic Polyester straps are 32 mm wide composed of 27 filaments of 1.1 mm diameter each. The straps are supplied by the manufacture in rolls of 455 mm diameter. The synthetic strap has linear stress-strain behaviour up to failure with nominal tensile strength of 570 MPa. The straps mainly used for strapping merchandise for shipment and storage, are inexpensive (0.3$/meter), and easy to handle and store. The straps have high strength-to-weight ratio as weigh approximately one seventh the weight of steel straps of the same dimensions. Moreover, they have good thermal properties as they can withstand a temperature range of -60 °C to +150 °C without any damage (Caristrap Inc., 1996). Figure 5.7 shows the composite straps used in the present experimental investigation.

A technical appraisal of **Caristrap Weatherguard** non-metallic Polyester Cord Strapping is given by the manufacturer. They defined Weatherguard as a non-woven synthetic material made of high tenacity space-age polyester fibers. These fibers are bonded together in parallel, the modulus of elasticity is designed to be highly oriented in

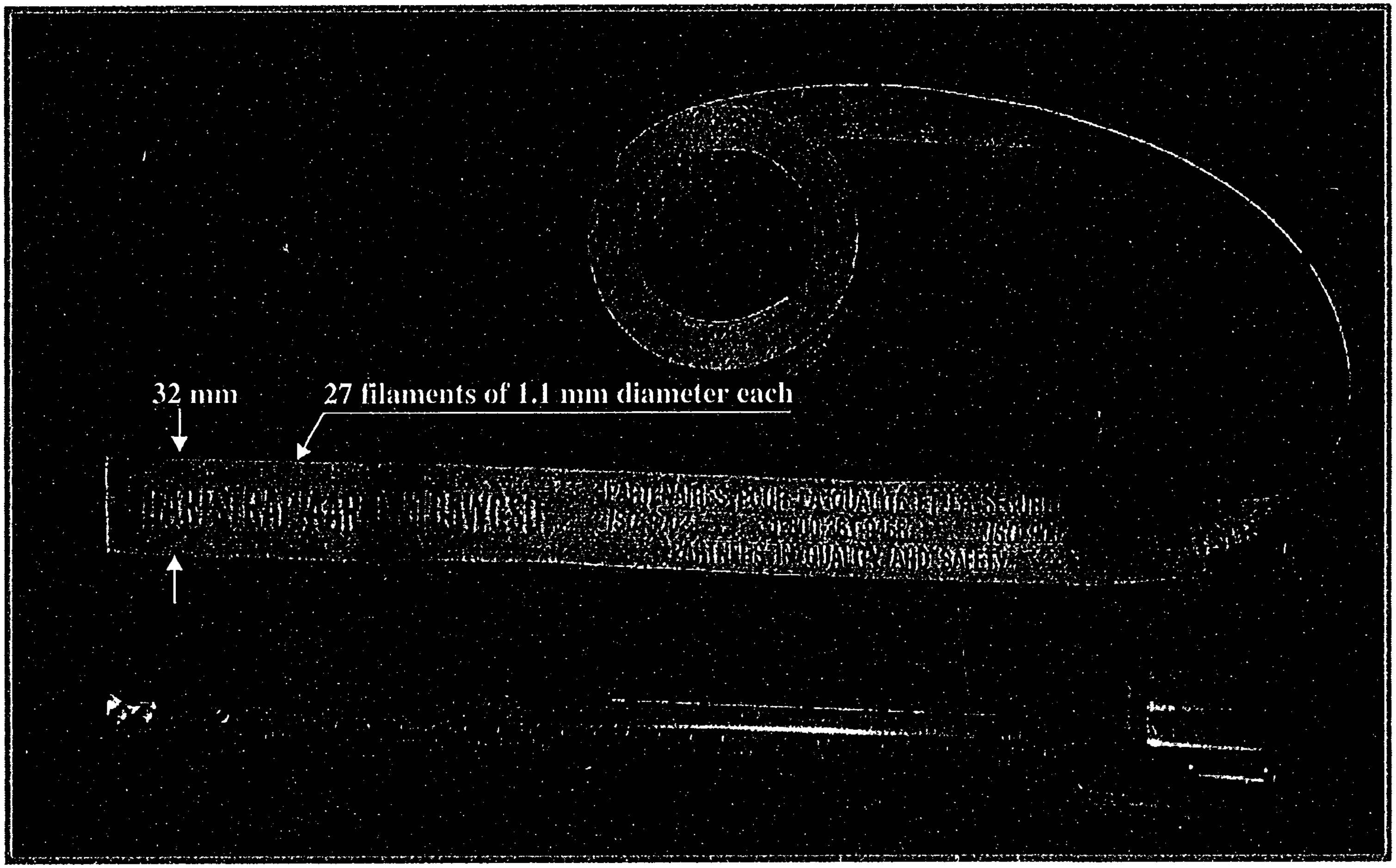


Figure 5.7 Composite Straps

the longitudinal direction. The strength of Weatherguard is in a direction proportional to the number of fibers and/or the thickness of each individual fiber used to make up the strap. The material is able to perform remarkably due to a combination of factors including the high tensile strength of the raw material, its shock absorbing elasticity and a resulting energy-to-break which is at least 2 to 3 times greater than steel strapping.

5.7.9 Adhesive

The Fast Weld Epoxy resin and hardener No.10 distributed by MF Composites Inc. and manufactured by Ciba-Geigy Inc. was used for this study. The two components of the high modulus high strength epoxy are to be mixed with equal ratios. The manufacture specified curing time for this type of epoxy to be 15 minutes at room temperature

5.7.10 Procedure of Repair

Two layers of straps were applied to the top surface of the ties to control the cracks resulting from the rebound forces. Before applying the first layer of the straps, it was important to prepare the surface of the concrete ties to create optimal adhesion conditions to ensure a strong bond between the composite material straps and the concrete surface. For this purpose, the top surface of the crossties was sand-blasted until the aggregate was exposed. This was followed by degreasing to remove any contaminants to achieve a proper adherent surface. The surface was then washed with water to remove dust and the fine particles were removed using compressed air. After the straps had been cut to the required size, the two components of the epoxy were hand mixed and a uniform 2 mm thin layer of the resulting paste was applied followed by installing the straps on the top surface of the tie. The straps were pressed onto the surface until the glue hardened. Pressing of the straps was done using a hard roller, in such way that the still fresh adhesive is squeezed out on both sides. This guaranteed that there remained no hollow spots. The straps were applied between the two supports to repair the cracked area. The composite of adhesive and straps was left for about an hour to allow for the resin to cure completely. The same procedure was then repeated for the second layer of straps. Figure 5.8 shows the two layers of straps installed on the top side of the crossties. After full curing of the adhesive, the crossties were re-instrumented to monitor their behaviour during reloading after repair. The upward loads were increased gradually at this stage from zero to failure of the ties in increments of 10 kN. After each load increment, measurement of midspan deflection and strain gauge readings were recorded. Also, the location of cracks and their lengths were clearly marked on the tie by a red marker to make the propagation of cracks after repair distinct from that on the tie before repair. The widths of cracks were also measured.

5.8 Experimental Results

The results obtained from the tests conducted on the crossties before application of the straps are compared to those from tests on the ties repaired using the epoxy-bonded composite straps. The comparison is extended to include the results obtained from an extensive investigation conducted by Peters (1992) on the behaviour of precast prestressed concrete crossties without straps.

5.8.1 Load-Deflection Response

The behaviour of each tie was characterized by its load versus deflection response to failure. Figures 5.9 and 5.10 depicts the load-deflection histories of crossties T1 and T2 before and after repair.

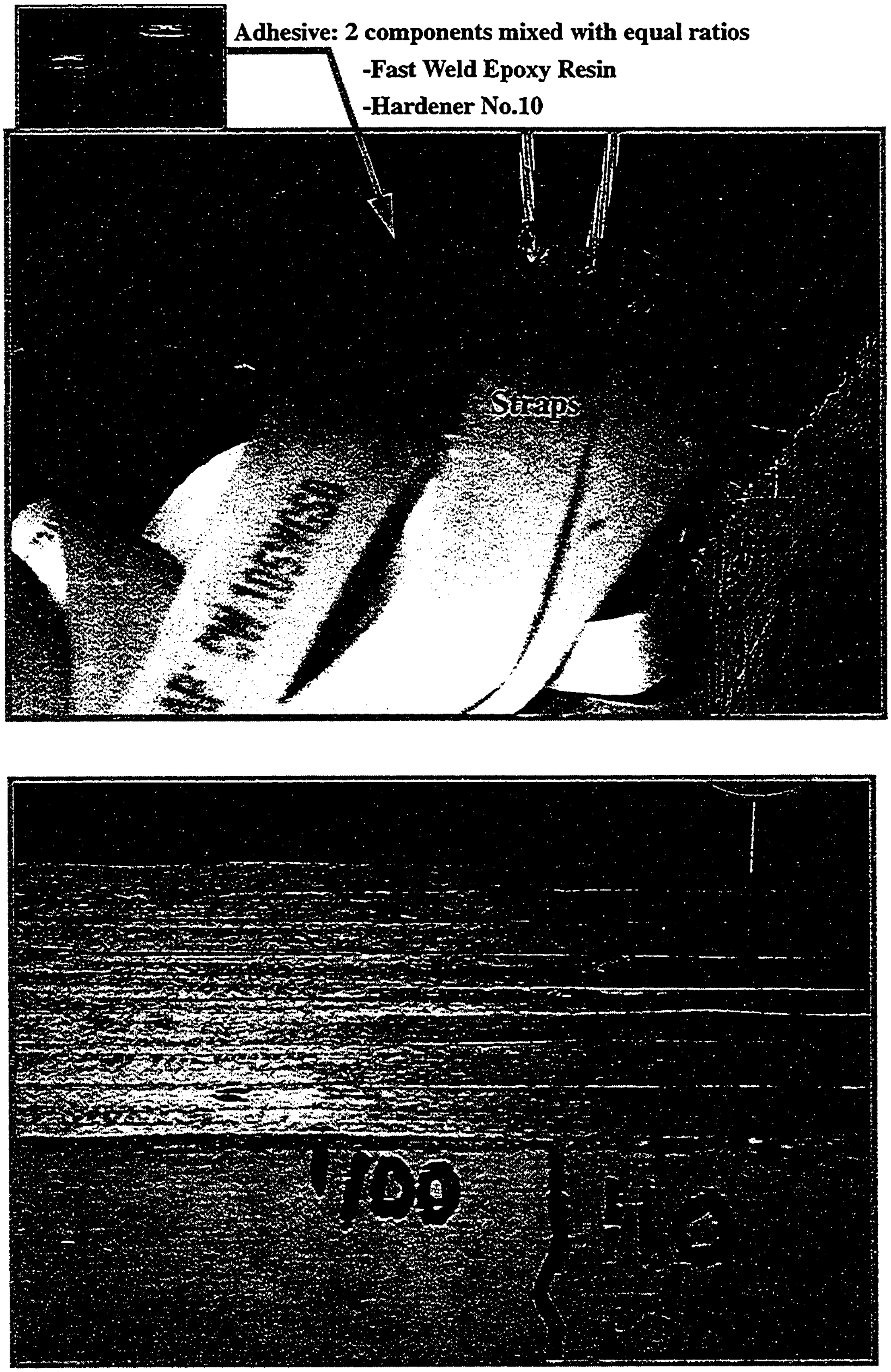


Figure 5.8 Installing the Two Layers of Straps on the Top Face of the Tie

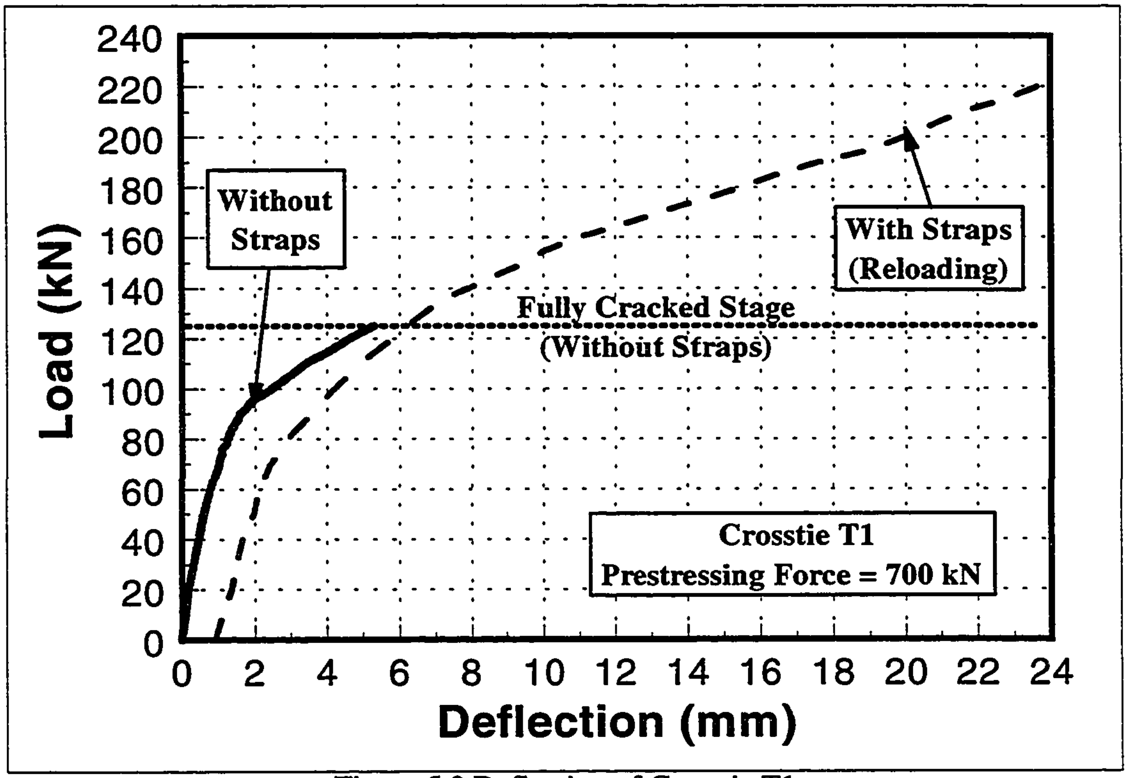


Figure 5.9 Deflection of Crosstie T1

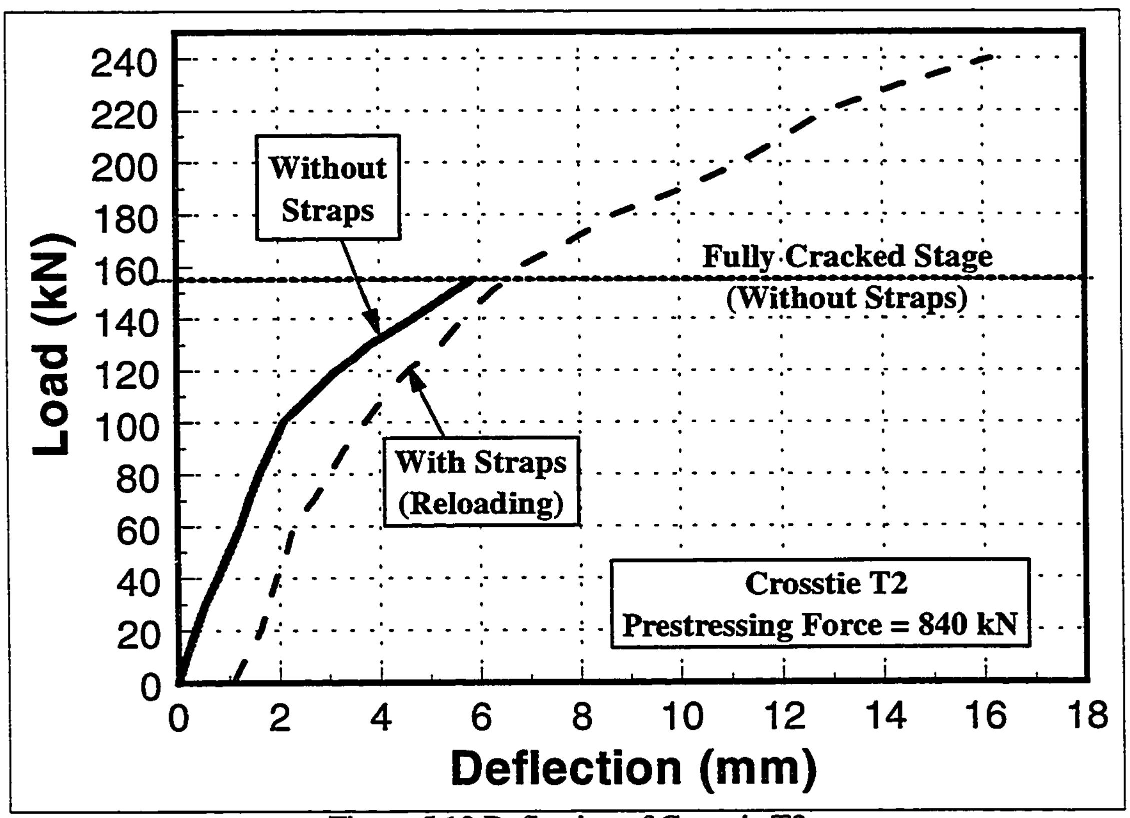


Figure 5.10 Deflection of Crosstie T2

For the first case of loading (without straps) the first cracks were obtained at load levels of 80 kN and 100 kN for T1 and T2, respectively. The increase in the cracking load in T2 was due to the higher prestressing force compared to T1. The load was increased up to a value of 125 kN for T1 and 155 kN for T2, and then gradually removed. These loads represent 1.55 times the load at first cracking. The loads were removed at this stage when the crack pattern in the constant moment region was fully developed and reached the stabilized state. At this stage, the deepest crack propagated to almost two-thirds of the total depth of the crosstie. Figures 5.11 and 5.12 show the crack patterns of crossties T1 and T2 without straps and just before removing the load.

After removal of the load, residual deflections of 0.92 mm and 1.07 mm were observed at midspan of T1 and T2, respectively. For the second case of loading (after repair with straps), the ties exhibited a linear deflection behaviour with slightly lower stiffness than in the first case of loading up to 80 and 100 kN, the loads at which the first cracking took place in ties T1 and T2, respectively without straps. Beyond these loads (during the reloading) when the original cracks tried to reopen, the straps stretched, acting as prestressing reinforcement at the top surface applying compression force on the top fibres of the ties. This force resulted in a balancing internal moment resisting the moment due to the externally applied loads. This increased the stiffness of the ties compared to that of the first case of loading (without straps). Crack spacings, widths and depths of penetration were recorded for the tested ties. The maximum crack widths at a load of 155 kN (fully cracked stage) for crosstie T2 before and after repair were 0.60 mm and 0.33 mm, respectively. The reduction in the crack width during reloading was due to the effect of the straps. It was observed that the straps did not debond prior to failure in any of the tests. Also no additional cracks were observed during the reloading stage of T1 and T2 up to the failure load.

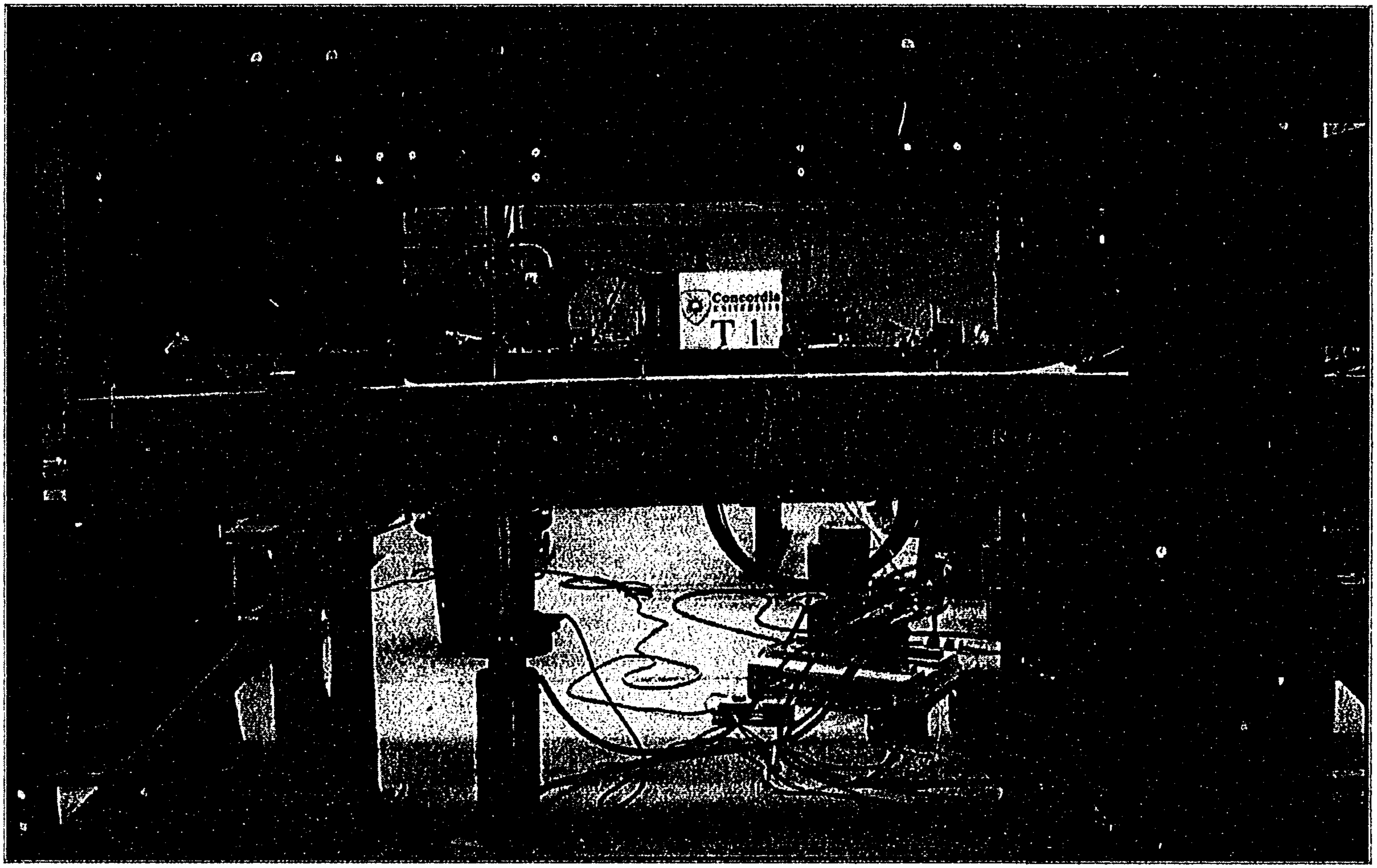


Figure 5.11 Crack Pattern of Crosstie T1

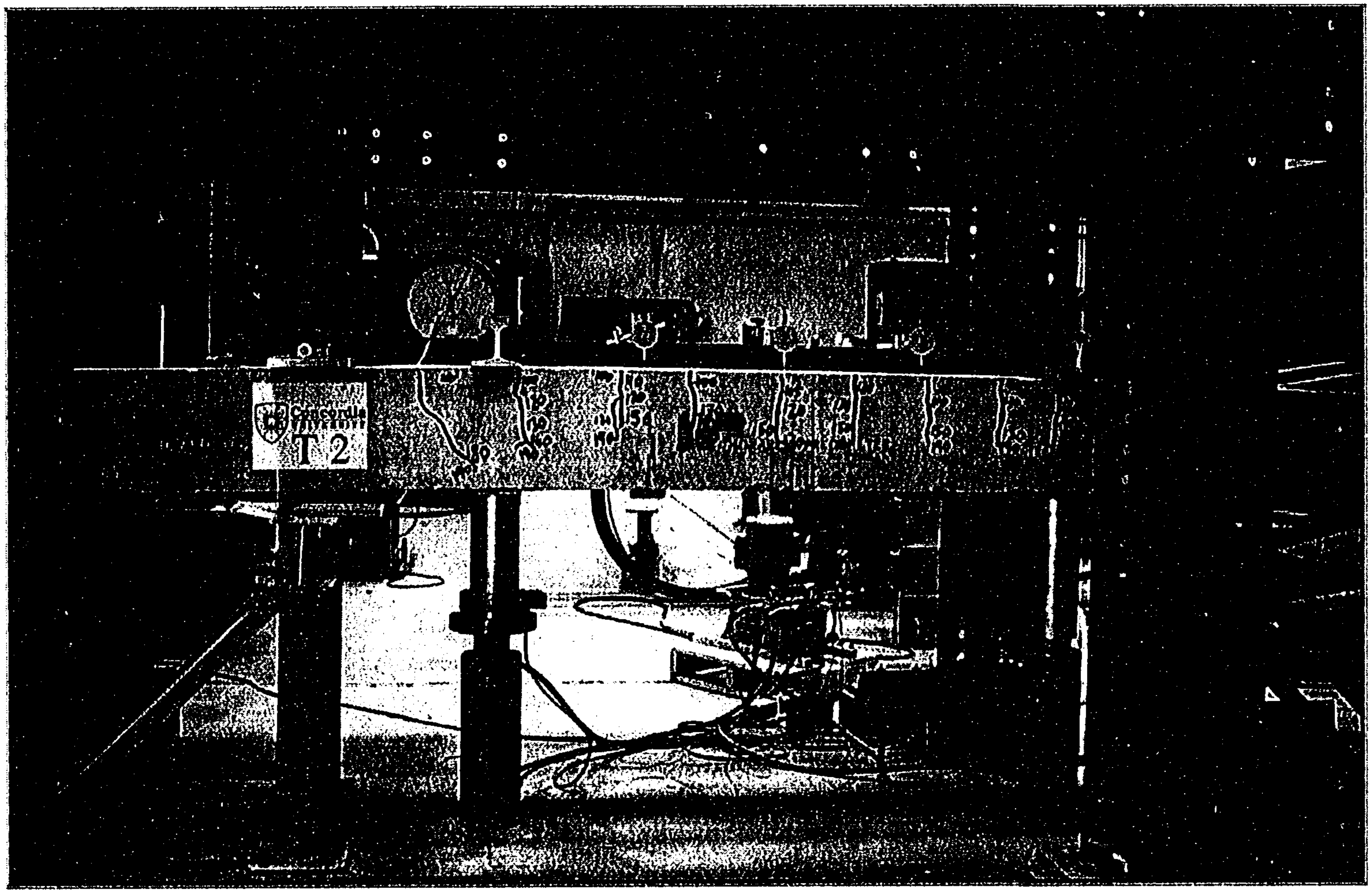


Figure 5.12 Crack pattern of Crosstie T2

5.8.2 Load-Strain Response

Concrete compression strains at the bottom fibers of a cross-section at midspan of crossties T1 and T2 before and after the repair with straps are shown in Figures 5.13 and 5.14 respectively. During reloading, the ties first exhibited higher strains compared to the first case of loading. This continued up to the cracking load where the slope of the load-strain curve was less than that in the first case of loading indicating higher stiffness for the repaired tie. This is attributed to the composite effect of the straps as described earlier.

5.8.3 Ultimate Capacity and Modes of Failure

The results of ultimate loads and modes of failure for crossties T1 and T2 strengthened with composite straps were compared to those of similar ties tested under the same conditions without strengthening. Figures 5.15 and 5.16 show the modes of failure for T1 and T2, respectively.

Crosstie T1 failed on flexure due to the concrete crushing at the compression zone (the bottom face) right above the left concentrated load as shown in Figure 5.15. The experimental ultimate failure load was 221 kN. This value is 40% higher than the ultimate flexural load for similar crossties tested monotonically from zero to failure without repair.

Crosstie T2, with two more top strands than in crosstie T1, failed on shear. Tie T2 behave similar to tie T1, the cracking pattern commenced with first cracking which led to uniformly distributed flexural cracking, except that with further increase of the load, diagonal cracks began to appear between the load point and the support (shear span), shortly thereafter the tie failed. Diagonal shear cracks appeared just prior to failure occurring. The flexural shear crack started at the top surface of the tie in the shear zone and propagated on 45° as shown in Figure 5.16. The measured shear failure load for T2 was 240 kN. This value is 48% higher than the flexural ultimate load recorded for similar crossties tested without straps.

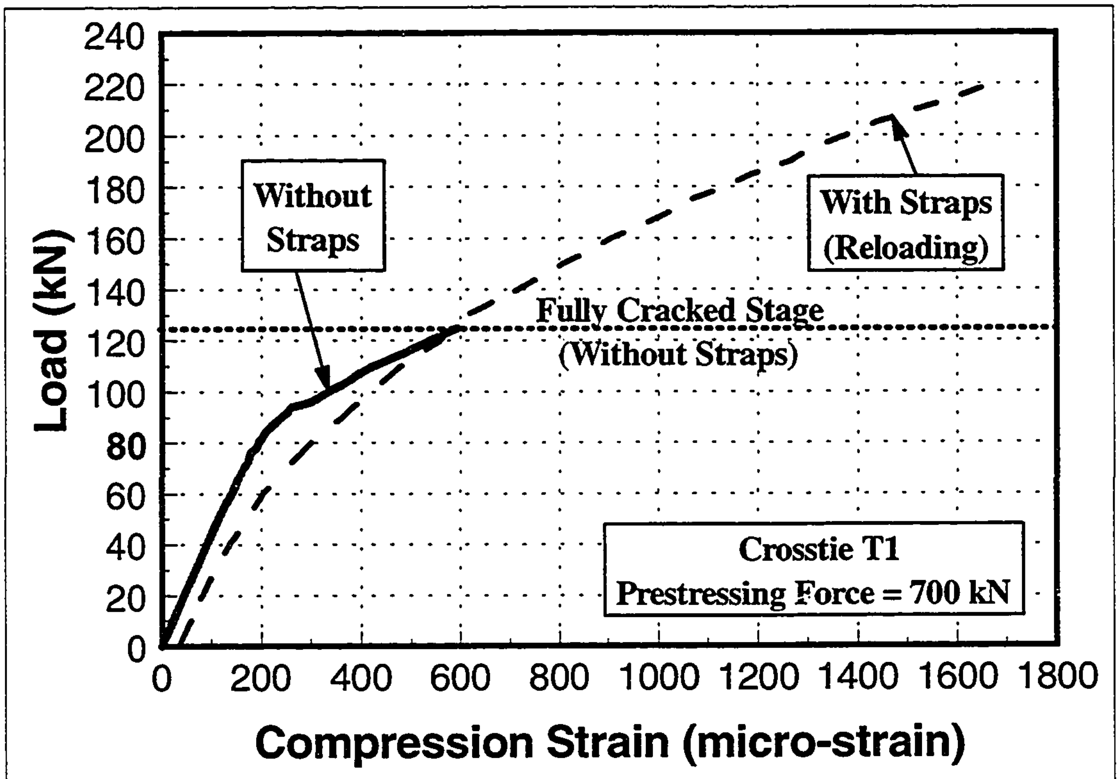


Figure 5.13 Concrete Compression Strains for Crossties T1

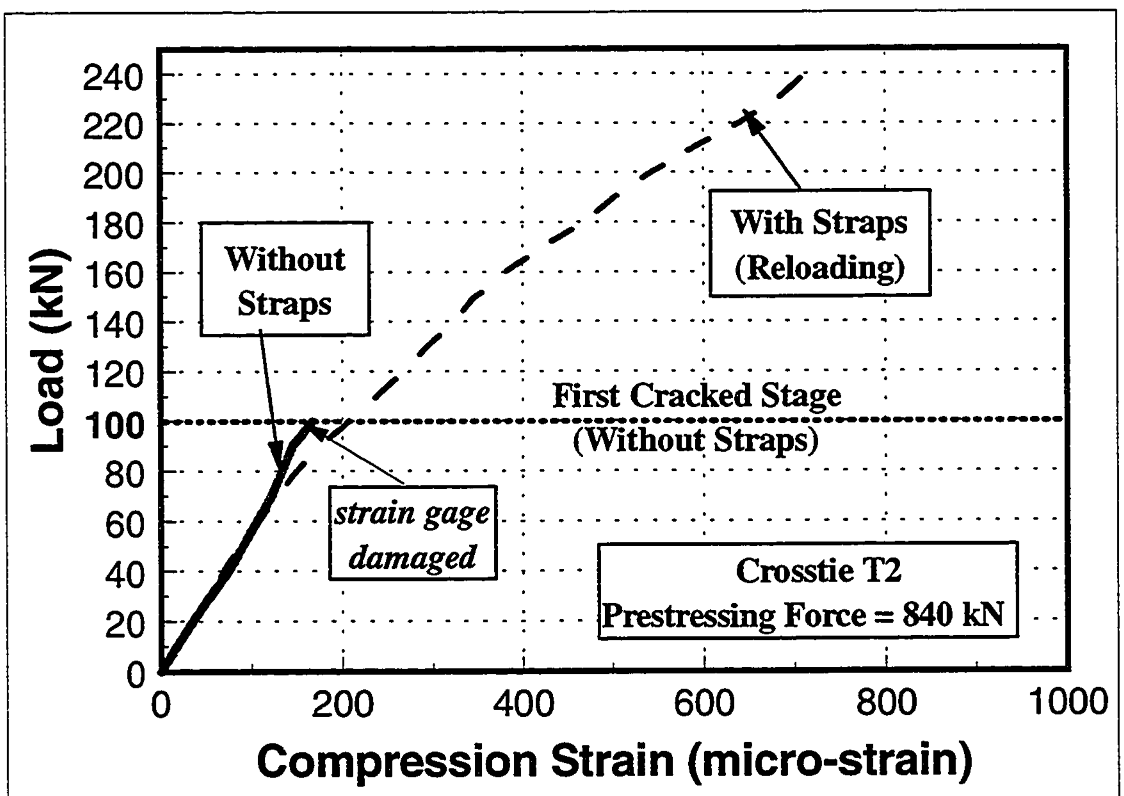


Figure 5.14 Concrete Compression Strains for Crossties T2

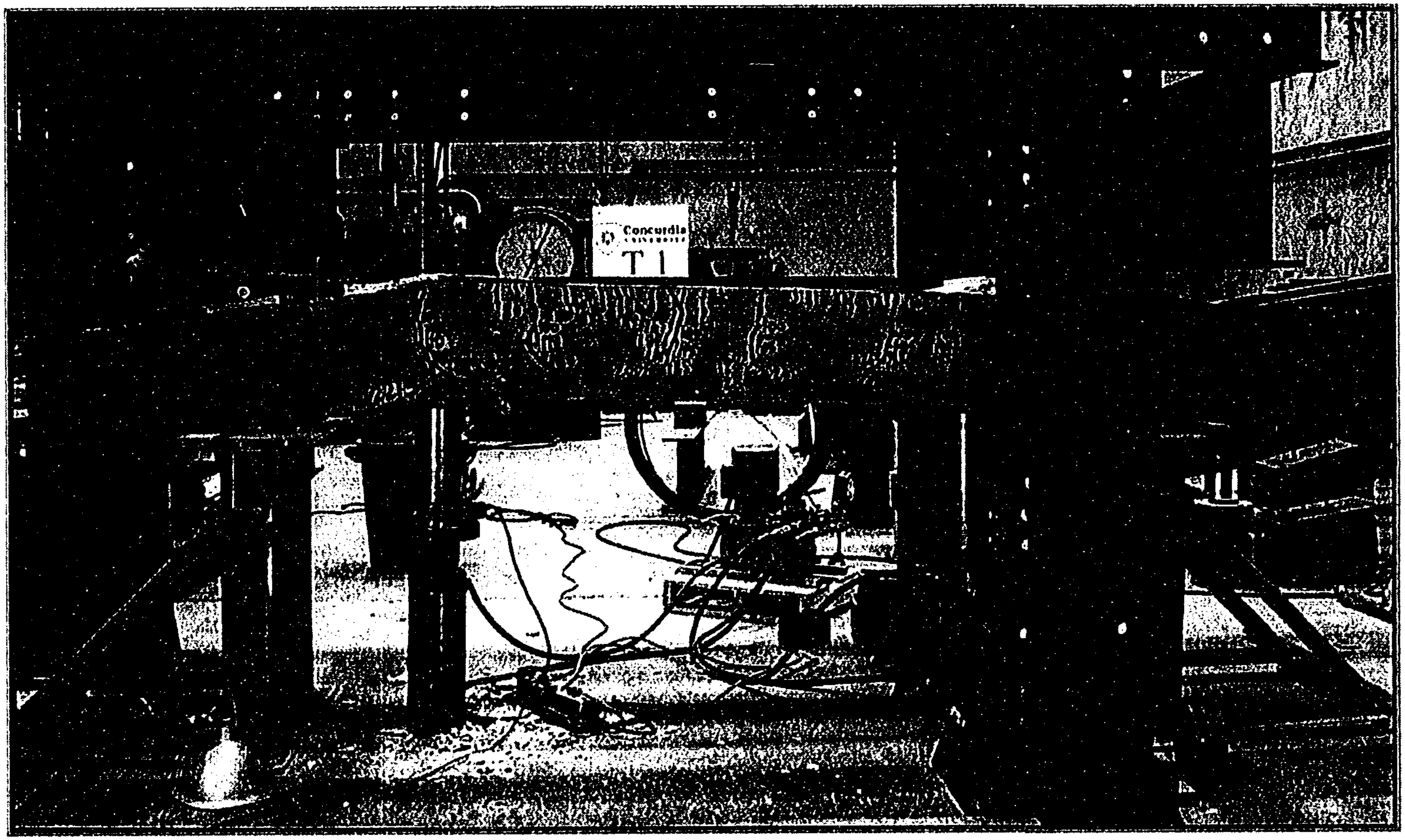


Figure 5.15 Crack Pattern and Failure Mode for Crosstie T1

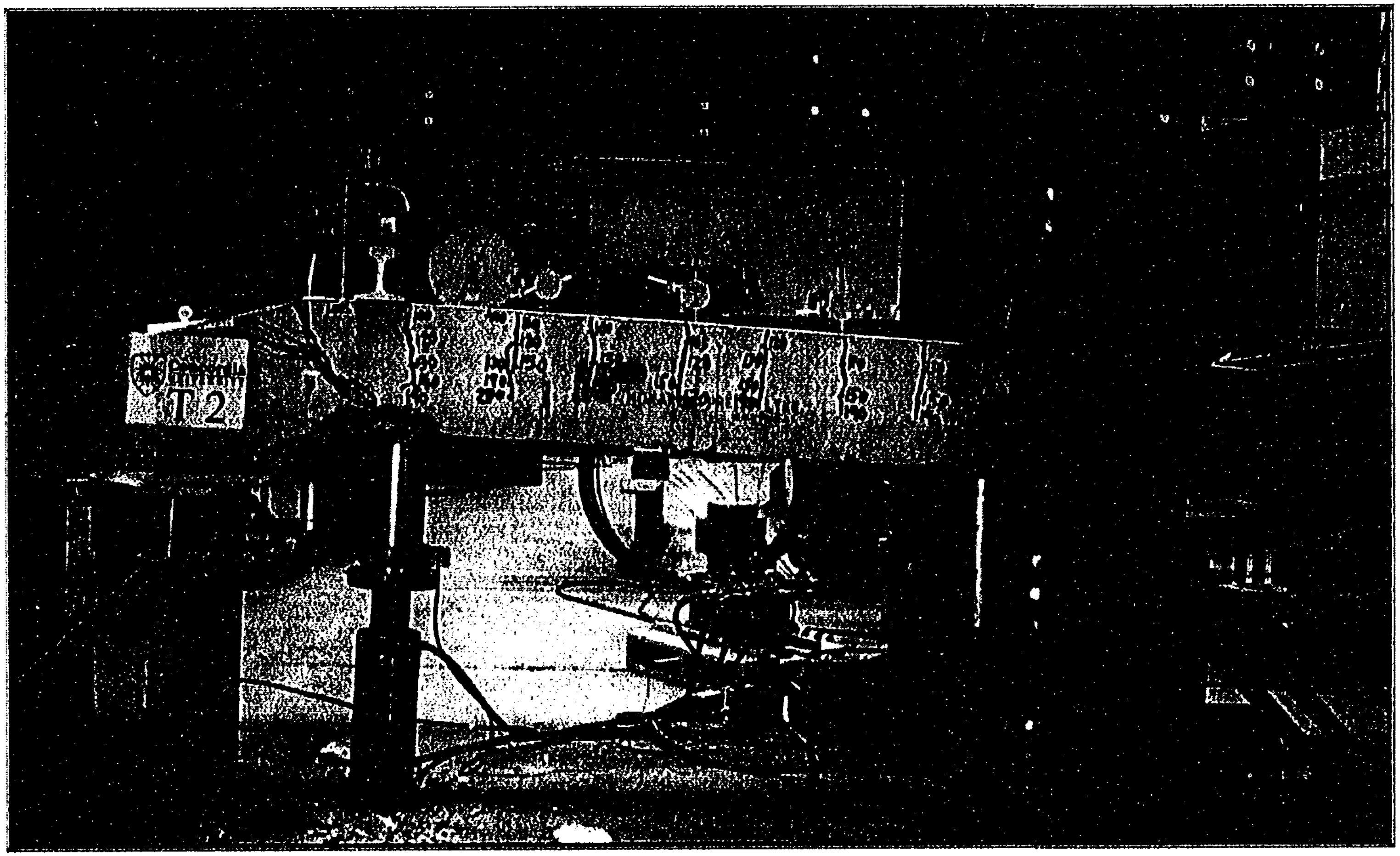


Figure 5.16 Crack Pattern and Failure Mode for Crosstie T2

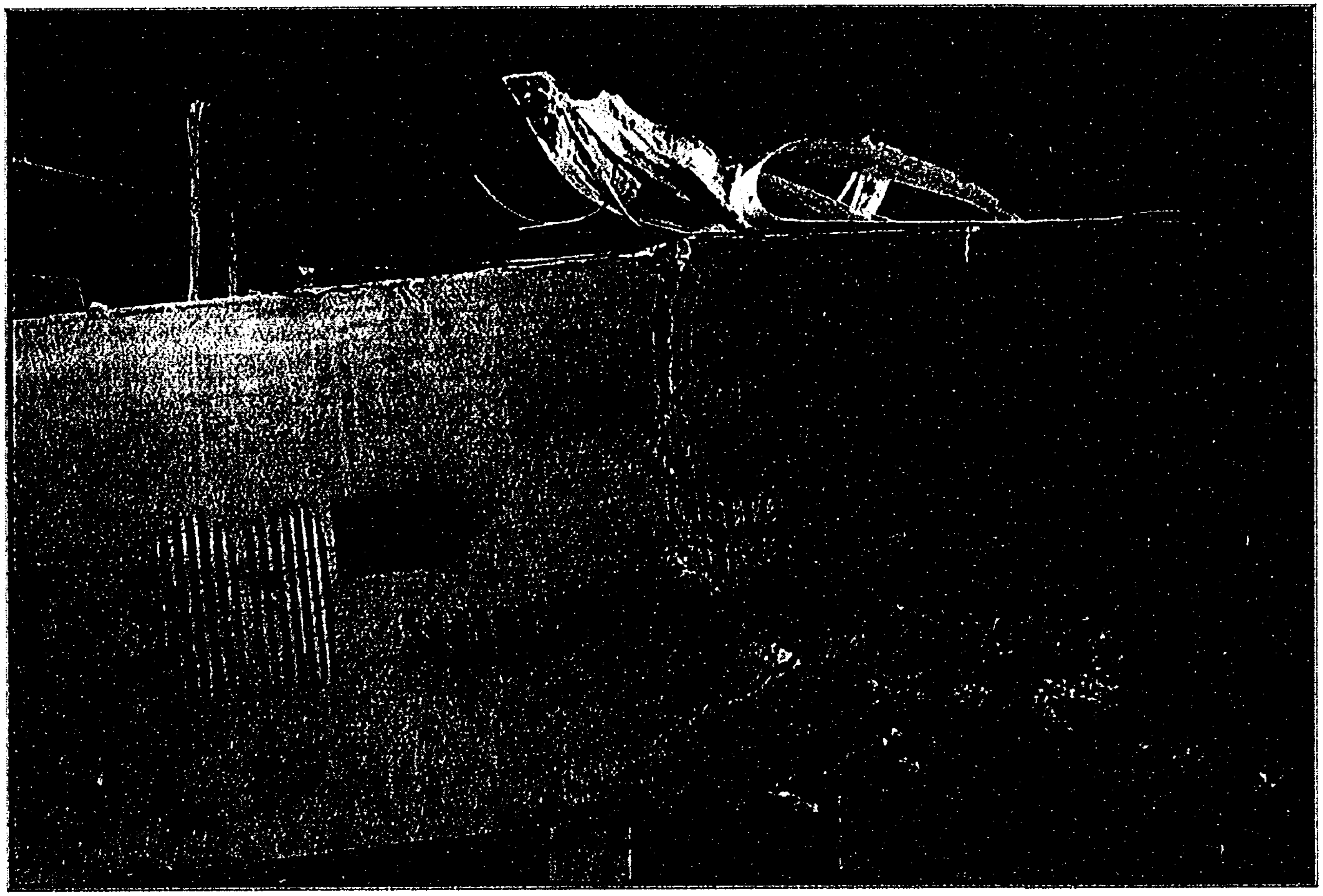


Figure 5.17 Close Up View of the Failure Crack for Crosstie T1

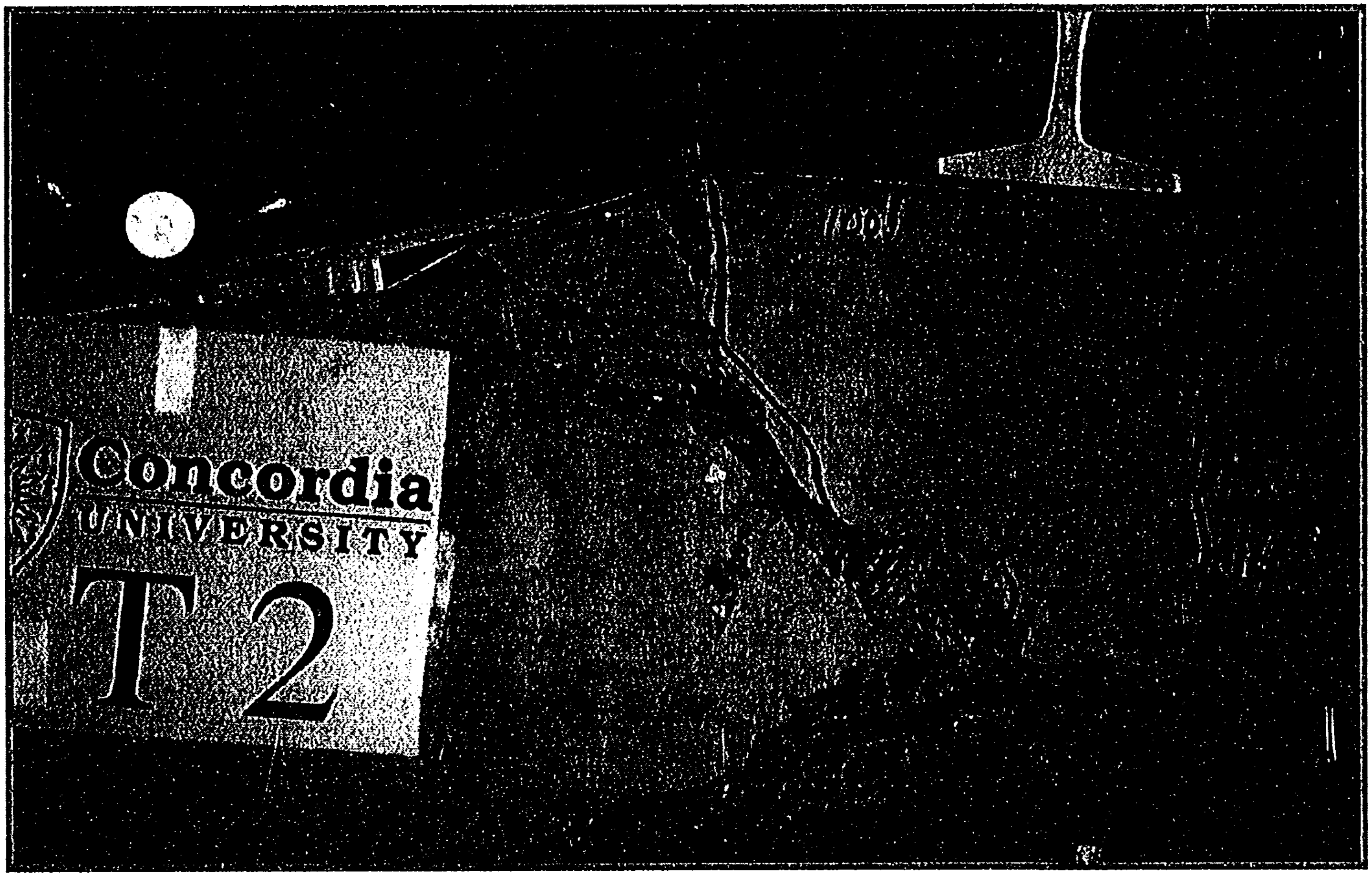


Figure 5.18 Close Up View of the Failure Crack for Crosstie T2

5.8.4 Effect of Development Length of Straps

In tie T1, the straps were applied between the supports. At failure, and due to the excessive curvature of the tie, failure in bond within the epoxy layers was observed. This failure resulted in continuous peeling-off of the straps as shown in the close-up view in Figure 5.17.

In tie T2, the straps were extended beyond the centre line of the support for a distance equal to the depth of the tie. This kept the straps bonded to the tie up to the failure load. After the shear failure had occurred, little separation between the straps and the epoxy was noticed above the shear crack due to the excessive curvature at this location. This is shown in the close-up view of failure crack of crosstie T2 in Figure 5.18.

5.9 Analytical Study

The nonlinear finite element program ABAQUS version 5.5 (Hibbitt et al., 1995) was used for the theoretical study. The crossties were analyzed without straps under two concentrated loads up to failure. A beam element with trapezoidal cross-section was used to represent the concrete. Rebars were modeled as one dimensional bar elements. In ABAQUS, the load is applied gradually in small increments. The size of each increment depends on the convergence of the iterative process in the previous increment of loading. The concrete under compression is modeled by an elastic-plastic theory, using a simple form for the yield surface expressed in terms of the equivalent pressure stress and the Von Mises equivalent deviatoric stress. Isotropic hardening is accounted for. Cracking is assumed to occur when the stress reaches the failure surface represented by a simple Coulomb line in terms of the equivalent pressure and the Von Mises equivalent deviatoric stress. The model used is a smeared crack model in the sense that it does not track individual macro cracks. Instead, constitutive calculations are performed independently at each integration point of the finite element model, and the presence of cracks is accounted for through the changes in the stresses and material stiffness associated with the integration point. The concrete behaviour is characterized by a stress-displacement response, in which the state of stress along the crack is a function of the opening strain across the crack.

Figures 5.19 and 5.20 show the finite element results for crosstie T1 and T2, respectively. The experimental results for the ties were included in the figures for comparison. Good agreement can be seen between the theoretical and the experimental results for the ties without straps. Comparing the finite element results with those during the reloading, it can be seen that the straps increased the flexural capacity and reduced the deflection of the crossties. The experimental results are very encouraging. It has been demonstrated that bonding of synthetic straps to the tension side using epoxy resin is indeed a feasible technique for upgrading the strength, stiffness and increasing the load carrying capacity of prestressed concrete crossties.

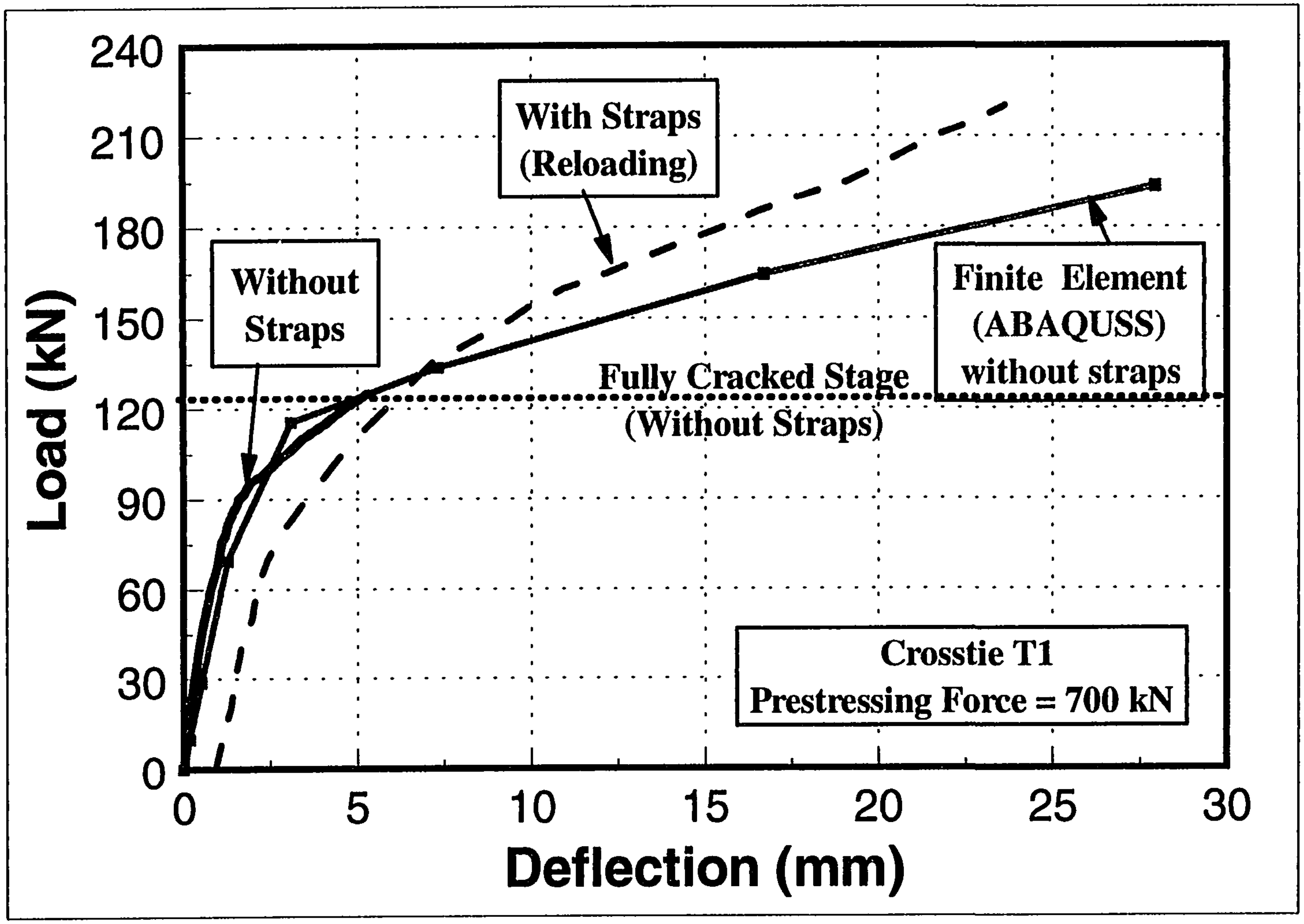


Figure 5.19 Experimental versus Analytical Results of Crosstie T1

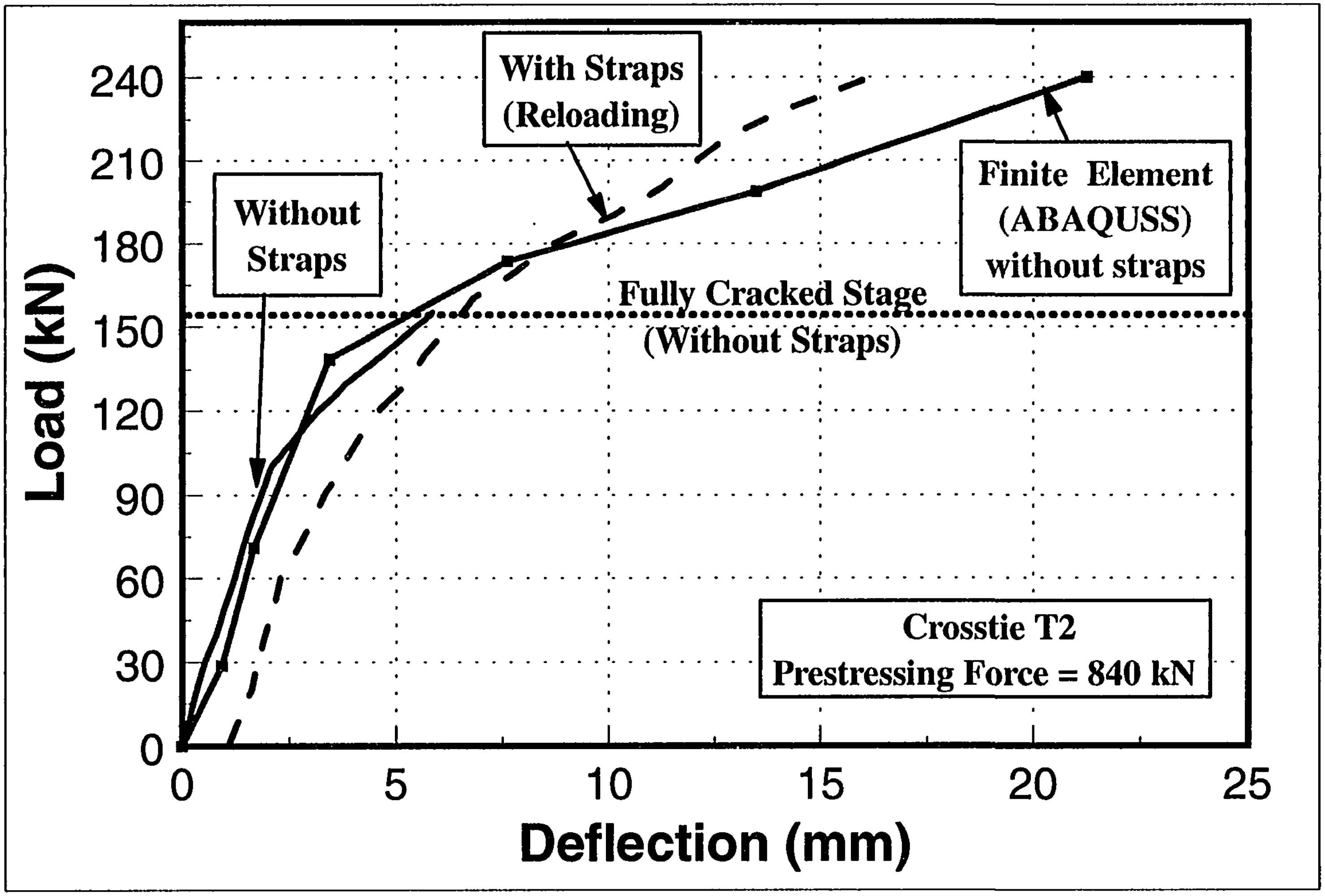


Figure 5.20 Experimental versus Analytical Results of Crosstie T2

Chapter 6

Summary, Conclusions, and Recommendations

6.1 Introduction

The major emphasis of the experimental programs presented in this thesis was to investigate and examine the effectiveness of fiber reinforced plastics when used to strengthen and retrofit reinforced concrete and partially prestressed concrete members. To achieve this objective, two experimental projects were carried out. The first project is concerned with strengthening of concrete members using externally prestressed carbon fiber reinforced plastic cables. The second project investigates retrofitting of concrete members with epoxy-bonded composite straps.

Summary and conclusions of the two projects are presented in this chapter. Recommendations are also given for further research in the area of strengthening and rehabilitation of concrete structures using advanced composite materials.

6.2 Project 1: Strengthening of Concrete Members Using Externally Prestressed Carbon Fiber Reinforced Plastic Cables

6.2.1 Summary

The main objective of this research project was to investigate the effectiveness of strengthening existing concrete structures with externally post-tensioned FRP ropes, particularly those made of carbon fiber reinforced plastic, in lieu of conventional steel tendons. More specifically, the objective was to determine the increase in the ultimate moment capacity of damaged flexural members after being strengthened by externally prestressed carbon fiber composite cables (CFCC). Another major objective was to study the effects of important parameters such as the span-to-depth ratio (S/d_p) and the level of internal prestressing expressed in terms of the reinforcing index (ω) or the partial prestressing ratio (*PPR*) on the behaviour of the reinforced or prestressed concrete beams strengthened with the external CFCC cables. Particular attention was given to the effects of these two parameters on the change in stress in the external cables and on their efficiency in strengthening damaged partially prestressed members.

A total of twelve partially prestressed concrete beams were tested. The beams were divided into three groups; each group consisted of four beams of the same span-to-depth ratio but with different levels of internal prestressing. The beam span-to-depth ratio varied from one group to another and were 10.7, 17.85, and 25, respectively. All the beams were subjected to two symmetrical concentrated loads applied in small increments at the one-third points of the span. In each group, three beams with different prestressing levels were subjected first to loads high enough to produce considerable cracking and deflections. The magnitude of the total load applied in this stage differed from one beam to another depending on the span length and the amount of internal prestressing. Some of the beams were then left under their own weight for a period of time to experience some time-dependent deformations. The purpose of this stage was to simulate the actual conditions of concrete flexural members that require strengthening and/or rehabilitation. Afterwards, the external prestressing using the CFCC was applied and the beams were subjected to a monotonically increasing load up to failure. The fourth beam in each group was tested monotonically up to failure without strengthening and was used as control beam for comparison purposes.

The CFCC external cables used for strengthening nine of the beams consisted of 1×7 strands of 5mm or 7.5mm nominal diameter, and modulus of elasticity of about 137 GPa. The tendons were produced and supplied by Tokyo Rope Manufacturing Co. Ltd. and Toho Rayon Co. Ltd. in Japan. For all the externally prestressed beams, the CFCC were draped at midspan by means of a deviator, and the eccentricity of the cables at the anchorages was zero. Each beam specimen was externally prestressed using two cables, each one was located at 25 mm away from the side of the cross-section. The two cables were stressed simultaneously in small increments using two identical jacks both connected to the same hydraulic cabinet to ensure equal distribution of the total prestressing force to each cable until the desired prestressing force was reached. The target effective prestressing force in the cables was about 50% of the guaranteed breaking load.

All measurements such as beam deflections, the concrete strain at the top and bottom faces of the beam, strains in the reinforcing steel, the crack widths and spacings and the force in the external cables were recorded.

An analytical study was also conducted in this project to calculate the change in stress in the external cables under increasing applied load. A model proposed by Virlogueux for the change in stress in internal unbonded tendons was used. A new equation derived based on consideration of geometry of deformations of the externally prestressed beams was proposed. The results of Virlogueux's equation and the new equation were compared with the change in stress obtained from the measured force in the cables.

6.2.2 Conclusions

The results of the concrete beams strengthened by external post-tensioned CFCC cables were presented and discussed in Chapter 4 and Appendix A. The following conclusions were drawn from the current experimental study:

1. The load-deflection response was improved after strengthening, this was observed from an increase in the stiffness of the concrete members after strengthening and reloading. In addition, the ultimate deflection after strengthening was smaller in comparison with the ultimate deflection of the control beams without strengthening.

2. The test results demonstrated that external prestressing improves significantly the ultimate flexural resistance of the concrete members. An increase in the ultimate flexural resistance up to 70% was obtained using external prestressing to strengthen concrete members depending on the amount of internal prestressing (*PPR*), and the member span-to-depth ratio (S/d_p). The increase in ultimate resistance of the strengthened beams depends also on the level of prestressing introduced in the external cables at the time of strengthening and on the extent of damage in the concrete

members before strengthening. In the present experiments, only 50% of the guaranteed breaking load of the CFCC cables was introduced at the time of strengthening. The tested beams also experienced considerable cracking and deformations before strengthening. Application of a higher external prestressing force in the CFCC cables at an earlier stage of cracking and deformation in the tested beams would have led to an increase in the ultimate resistance of the strengthened beams higher than the 70% obtained in the present investigation.

3. The external prestressing was shown to reduce the crack widths or close the cracks completely, and lead to a stiffer load-deflection response.

4. The comparison between the results of Virlogueux's equation, the new proposed equation and the experimental values of the change in stress in the external prestressed cables indicated that Virlogueux's equation gives higher values for all beams and is therefore more conservative than the proposed equation. The latter, however, gives values of stress change close to the measured values particularly at ultimate for most of the beams. Until further research is available, the proposed equation can be used to estimate the change in stress in the external cables.

5. The depth of internal prestressing d_p was kept constant while the span length was increased to produce different S/d_p ratios. The experimental results indicated that increasing the span-to-depth ratio of the beam results in an increase in the change in stress Δf_{pf} in the external post-tensioned CFCC cables. Also, increasing the span-to-depth ratio (S/d_p) increases significantly the resistance of the strengthened beams.

6. Increasing the partial prestressing ratio (*PPR*) and the reinforcing index (ω) in the beams with the same span-to-depth ratio increases the ultimate flexural resistance. Also, because of the larger value of the tension reinforcement provided in the partially prestressed concrete beams compared to the minimum value required by the code, the flexural stiffness of the beams increased with increasing the reinforcing index.

7. It can thus be concluded that external prestressing using CFCC cables is a very powerful technique for strengthening or rehabilitation of concrete structures.

6.2.3 Recommendations for Further Research

The present investigation was limited to some but important parameters that affect the behaviour of concrete members strengthened with externally prestressed cables made of fiber reinforced plastic materials. Further research is needed to investigate the effects of other parameters and to provide a comprehensive understanding of the behaviour of structural concrete members under this type of strengthening technique. Some of the investigations needed in this area are recommended below:

1. The effect of varying the level of the initial external prestressing and the extent of damage in the concrete member at the time of strengthening on the ultimate load capacity and the serviceability of the strengthened members need to be investigated.

2. Tests need to be carried out to study the enhancement of serviceability and ultimate strength under fatigue and cyclic loading of concrete members strengthened with external FRP cables.

3. In the present investigation, only one profile (a single-point draped) of the external FRP

cables was considered. The effects of other profiles such as straight and two-point draped cables need to be investigated. The effects of variation of eccentricity of the external cables with the change in load level and time-dependent deformations should also be studied, particularly for members with large span-to-depth ratios.

4. More research is required to investigate the efficiency of the use of external FRP cables as part of the primary reinforcement of the concrete members in improving the serviceability and the ultimate capacity of new structures.

5. Particular attention should be given to developing new and better anchorage systems of externally prestressed FRP cables. The use of fixed die-cast, particularly by those made of steel, should be avoided.

6.3 Project 2: Retrofitting and Strengthening of Prestressed Concrete Railway Crossties with Composite Fabric Straps

6.3.1 Summary

The purpose of this research was to study the feasibility of retrofitting prestressed concrete railway crossties by application of composite fabric straps bonded with epoxy adhesive to the top surface of the ties which can be subjected to cracking caused by rebond forces due to wheel impact. In the experimental program two full scale ties with different levels of prestressing and different arrangement of pretensioned strands in the compression zone were loaded gradually until the cracking pattern in the constant moment region was fully developed and reached the stabilized state. Two concentrated loads were applied from the bottom upward to simulate the rebound force from the railway vehicles. At the end of each load increment, the crack propagation was marked and measurements of midspan deflection, strain gauge readings and crack opening were recorded. The load was increased up to 1.55 times the load at first cracking; the stage at which the deepest crack propagated to almost two thirds the depth of the crossties. The applied load was then removed and the crossties were strengthened using composite fabrics straps placed on the top surface of the tie before re-testing up to failure. Two layers of epoxy-bonded 105WGSD Polyester fabric straps supplied by Caristrap International Inc. were used to improve the strength and serviceability of the ties. The synthetic Polyester fabric straps are 32 mm wide composed of 27 filaments of 1.1 mm diameter each, and have linear stress-strain behaviour up to failure with nominal tensile strength of 14.63 kN (570 MPa). The Fast Weld Epoxy resin and hardener No.10 distributed by MF Composites Inc. and manufactured by Ciba-Geigy Inc. was used for this study and mixed with equal ratios. The advantage of this strengthening technique is the ease of application with the favorable characteristics of being very light in weight for which the use of these straps becomes quite attractive.

6.3.2 Conclusions

The experimental investigation on the use of external strengthening in the form of FRP straps, bonded to the tension face of the prestressed concrete crossties led to the following conclusions:

1. Strengthening of prestressed concrete crossties with composite fabrics is feasible, efficient and cost-effective.
2. A significant increase in the strength can be achieved by bonding of composite fabrics to the tension face of the prestressed concrete crossties. Furthermore, the strength of the

repaired crossties was increased from 40% to 50% than that of the crossties tested monotonically up to failure without repair.

3. The synthetic fabrics resisted reopening of cracks during the reloading after repair. The stiffness of the repaired ties beyond the cracking load was higher than that of the original ties before repair. The crack widths were smaller in the repaired ties than those in the ties without repair.

4. At ultimate, because of the excessive curvature of the tie, failure in bond within the epoxy layers leading to continuous peeling-off of the fabric straps was observed. Particular attention should therefore be given to the anchorage length of the straps at their ends.

5. A comparison of the finite element results for the ties without straps with the experimental results for the ties with straps indicated that the straps increased the flexural capacity and reduced the deflection of the crossties.

6. Careful preparation of the bonding surface and proper mixing and application of the epoxy adhesive are vital to the successful implementation of this strengthening technique.

6.3.3 Recommendations for Further Research

The Polyester fabric straps supplied by Caristraps International Inc. are mainly used for strapping merchandise for shipment and storage. Their use for strengthening of structural concrete elements is implemented for the first time in this experimental project and their application to civil engineering structures can not be established before several studies are carried out as suggested below:

- Since the increase in the flexural capacity of concrete members strengthened by application of the synthetic fabric straps depends on the cross-sectional area of the straps and hence the number of layers, a study on selection of the optimum number of layers or the cross-sectional area of the straps need to ne conducted.
- The long-term performance of the bond at the interface of the concrete and composite straps, including the effects of fatigue and unfavorable environmental conditions needs to be investigated. The effects of the epoxy resin on the strength of the straps and the use of different types of epoxies and composite straps to select the optimum combination of the two, and to maximize the performance enhancement of the two materials need to be considered.
- The effects of harch environmental conditions, such as wet/dry cycles using salt water on the performance of FRP bonded straps to concrete beams and on the interface bond between the straps and the concrete need to be investigated in order to establish the durability of the straps.
- The use of the straps as external reinforcement, or as prestressing reinforcement for new structural members can be considered.
- The feasibility of enhancing the shear capacity of concrete members retrofitted by means of the straps needs to be studied.
- Behaviour under sustained loading, thermal cycles, humidity cycles, fatigue loading, and dynamic loading, all these are required to be fully studied and investigated.

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Appendix A

Details of Experimental Results in the Beams Strengthened with External Prestressing

Details of the experimental results for the entire response history of the beams strengthened with external prestressing CFCC cables during testing are presented in Tables A.1 to A.9. The tables give the total applied load, deflection at midspan, stress in the tension reinforcement, concrete strain in compression, the cracking load, the average crack spacing, the total number of cracks, the cracking pattern and stabilization state, as well as the force in the external cable. The tables also include information and relevant observation regarding the initial damaging load, the crack distribution, the ultimate load and failure mode for each beam specimen. Typical photograph showing the crack distribution and a close up view of failure of each beam specimens are presented in Figures A.1 to A.31

Table A.1: Behaviour of Beam RCB1 during Testing

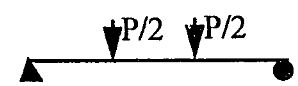
L = 2.00 m
S = 2.25m
$S/d_p = 10.7$
b = 150 mm
h = 280 mm
$d_s = 250$ mm
$d_{ps} = 210$ mm
$d_{pe} = 324$ mm

Ordinary Reinforcement:
A_s = 2#10M (200 mm^2)
$f_y = 400$ MPa
$E_s = 200000$ MPa
A_s' = 2-6mm plain bars (56.62mm^2)
$f_y' = 275$ MPa

External Prestressing profile:
one draped point at middle
Type (1×7) CFCC 5.0mm
$A_{pf} = 2(10.1) = 20.2mm^2$
$f_{puf} = 2120$ MPa
$f_{pef} = 0.5 f_{pu}$
$E_{pf} = 144000$ MPa
$f_c' = 35$ MPa (Design)

$f_c' = 44.57$ MPa (at time of testing)

Moment at midspan = PL/6



Type of loading	Total applied load: P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	14	1.28	n/a	109	250	n/a	3	-----	First crack (hair line crack).
	24	2.47		270	125		5	-----	Steel strain gauges were damaged.
	30	3.33		379	108		9	-----	Crack widths were not measured.
	40	4.96		521	118		12	-----	Max. damaging load. Cracks are mainly distributed in the region of constant moment, with maximum crack depth equal to 2/3 beam depth.
	0	1.28		109	118		12	-----	
Sustained up to 82 days	1	2	n/a	315	118		12	-----	Own weight only.

Table A.1: Behaviour of Beam RCB1 during Testing (cont'd)

Type of loading	Total applied load: P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
reloading before external prestressing	12	3.38	n/a	458	118	0.08	12	-----	
	38	6.12		728	118	0.3	12	-----	Cracks started to propagate.
	40	6.44		754	118	0.33	12	-----	
external prestressing operation	40	6.44	n/a	754	118	0.33	12	0	
	40	6.4		747	118	0.3	12	10	Cracks closed partially.
loading after strengthening	48	7.15	n/a	823	100	0.33	15	10	New cracks in the shear span.
	52	7.74		867	94	0.4	16	13	Slip crack formed, and the cracks propagated upward.
	62	9.4		993	94	0.45	16	17	Cracks stabilized.
	68	11.48		1118	94	0.8	16	22	Cracks propagated towards the point load.
	74	16.9		1480	94	1.25	17	37	Slip crack formed near the support
	80	23.49		1564	94	1.5	17	49	Beam failed in Shear.
releasing the applied load	62	23.59	n/a	1538	94	1.25	17	47	
	0	14.8		837	94	1.00	17	26	
releasing external prestressing	0	14.93	n/a	857	94	1.00	17	0	

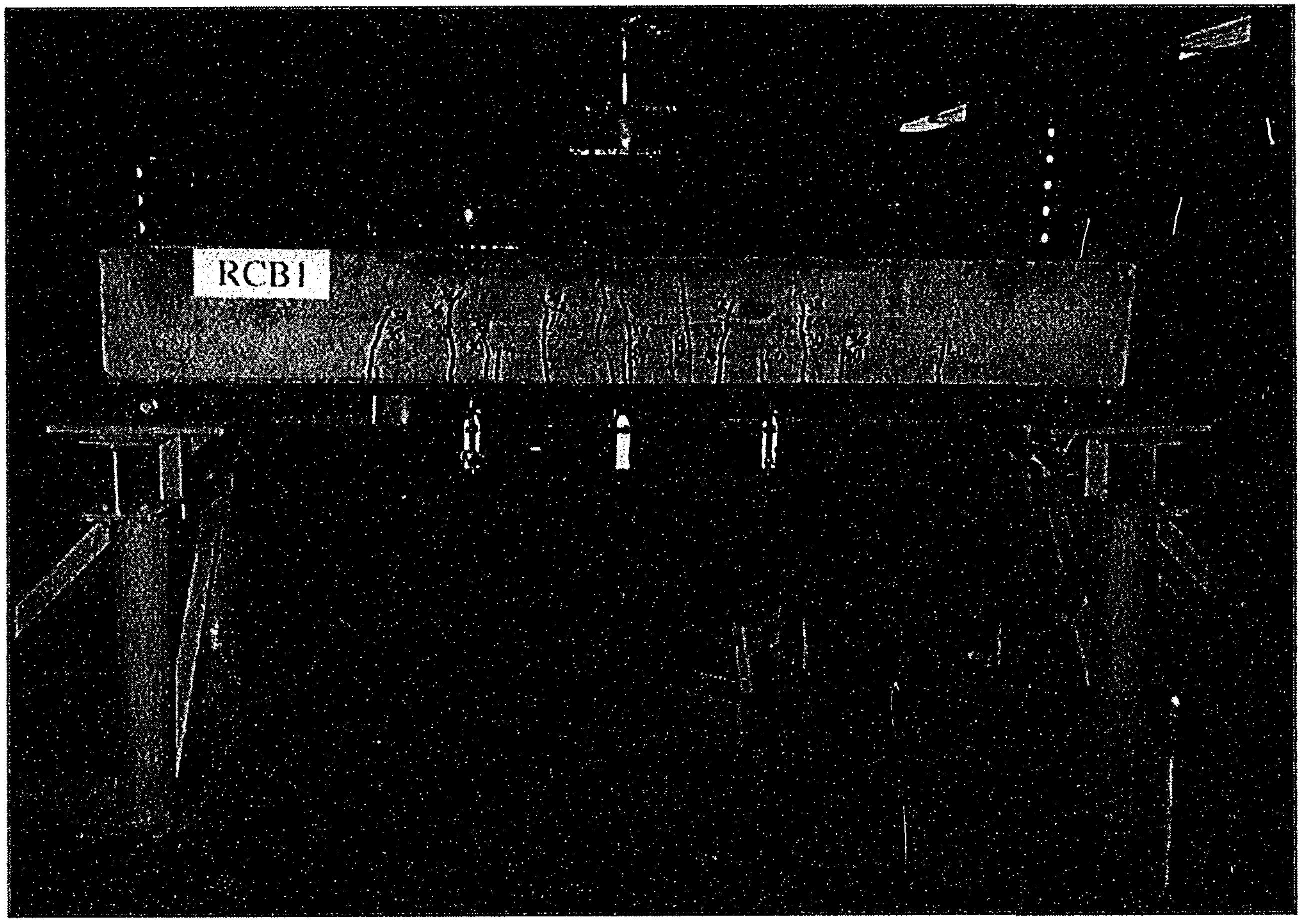


Figure A.1 Beam RCB1 after Application of Initial Damaging Load.

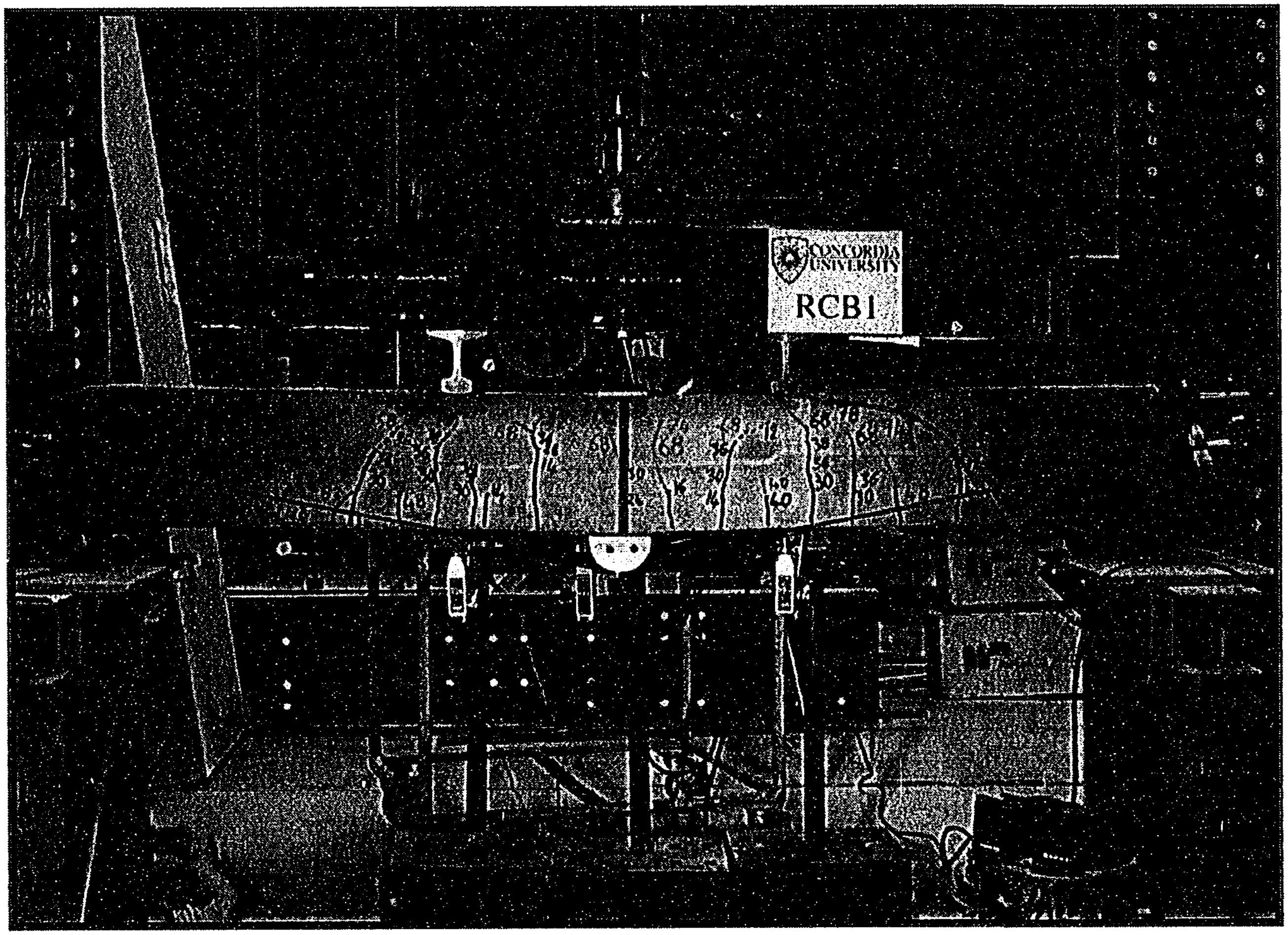


Figure A.2 Beam RCB1 at Failure

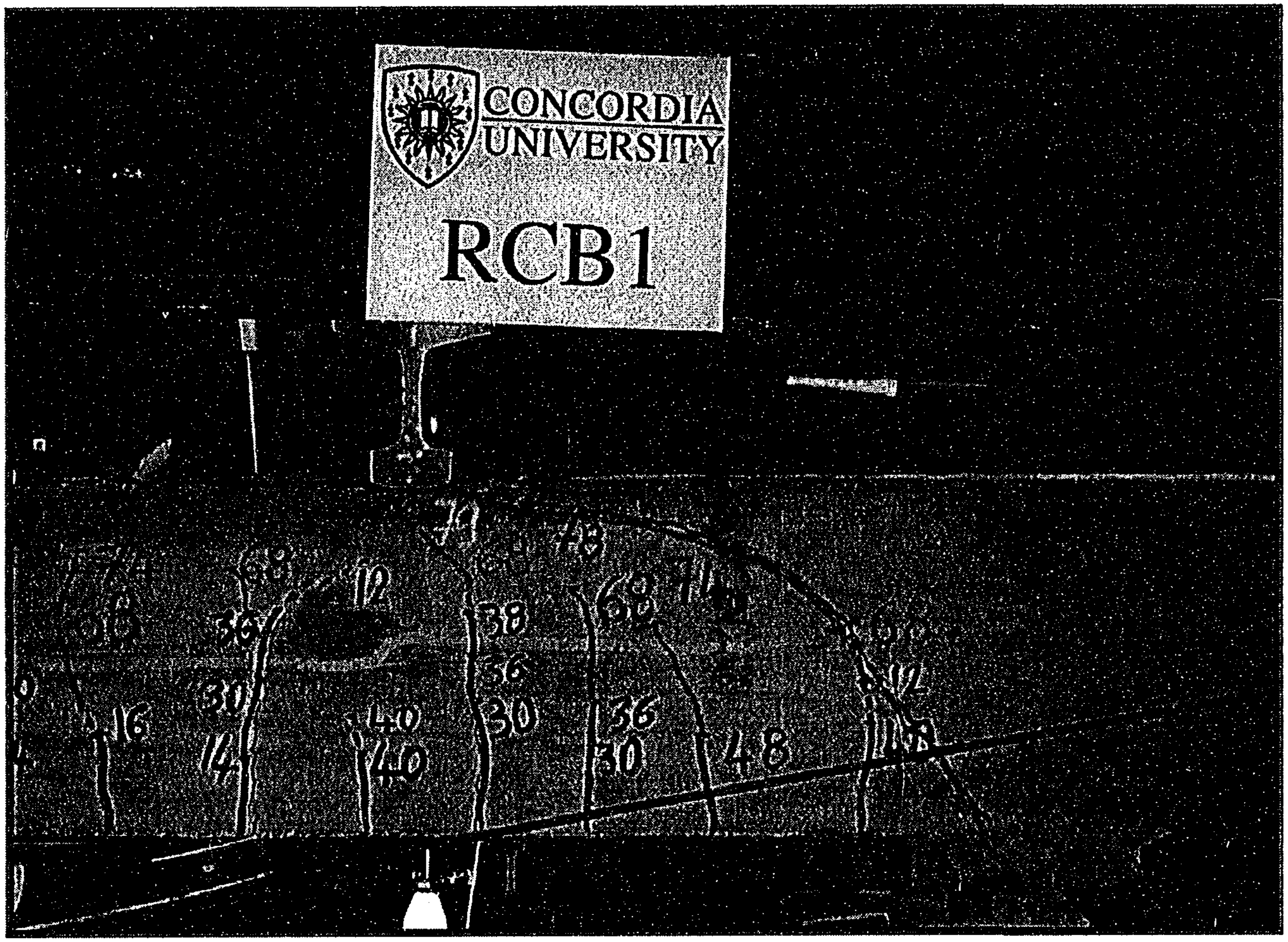


Figure A.3 Close-up of Failure of Beam RCB1

Table A.2: Behaviour of Beam PPCB1-1 during Testing

L = 2.00 m S = 2.25 m S/d_p = 10.7 b = 150 mm h = 280 mm d_s = 250 mm d_{ps} = 210 mm d_{pe} = 324 mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) f_y = 400 MPa E_s = 200000 MPa A_s' = 2-6mm plain bars (56.62mm^2) f_y' = 275 MPa	Internal Prestressing: A_{ps} = 1-3/8in strands (55.0 mm^2) f_{pu} = 1860 MPa $f_{py} = 0.9 f_{pu}$ (low relaxation) $f_{pe} = 0.6 f_{pu}$ E_{ps} = 200000 MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 5.0mm A_{pf} = 2(10.1) = 20.2mm^2 f_{puf} = 2120 MPa $f_{pef} = 0.5 f_{pu}$ E_{pf} = 144000 MPa	f_c' = 35 MPa (Design) f_c' 43.01MPa (at time of testing) Moment at midspan: M = PL/6

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	48	2.78	70.8	290	-----	h.l.	1	-----	First crack at midspan (hairline).
	54	3.35	112	383	200	h.l.	3	-----	New cracks formed.
	62	4.28	165	493	250	0.08	5	-----	All cracks formed in the region of constant moment and under the two point loads.
	64	4.5	175	517	190	0.1	6	-----	
	68	5.05	192.4	560	158	0.15	7	-----	Max. damaging load.
No time dependent effects. Beam was strengthened directly									
external prestressing	68	5.05	137	607	158	0.15	7	0	initial cable length = 2235 mm.
	71	4.78	106	553	158	0.08	7	10	0.27mm camber due to external prestressing..

Table A.2: Behaviour of Beam PPCB1-1 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
loading after strengthening	70	4.78	102.4	544	158	0.08	7	10	All cracks propagated equally at the same load levels. Almost symmetric cracking pattern under the two point loads. Cracks stabilized at P= 106 kN. Slip cracks started to form towards the supports. The max. crack depth =200mm.
	80	5.26	124.6	615	158	0.1	7	10	
	88	5.81	142.6	675	137	0.15	8	10	
	94	6.5	152	731	125	0.2	9	10	
	98	7.16	162.6	774	127	0.25	10	11	
	106	8.26	181	857	125	0.3	10	11	
	120	10.89	n/a	1027	125	0.8	10	12	Slip cracks developed. Max. crack depth = 230mm.
	128	14.39	n/a	810	125	1.5	10	12	Flexural shear failure. Cable length at failure =2260mm
releasing the applied load	0	13.94	n/a	46	125	1.5	10	10	
releasing external prestressing	0	13.83	n/a	79	125	1.5	10	0	

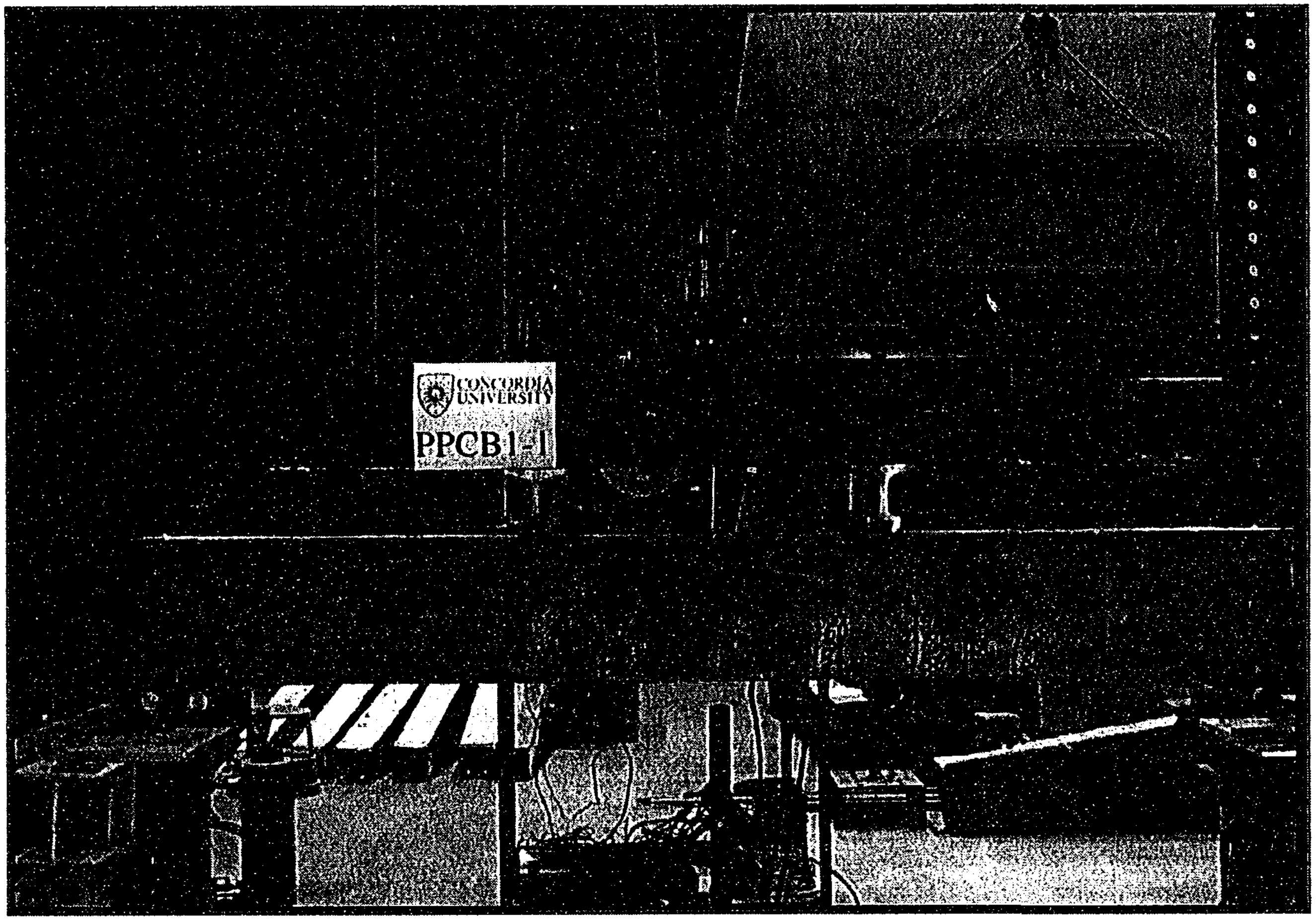


Figure A.4 Beam PPCB1-1 after Application of Initial Damaging Load

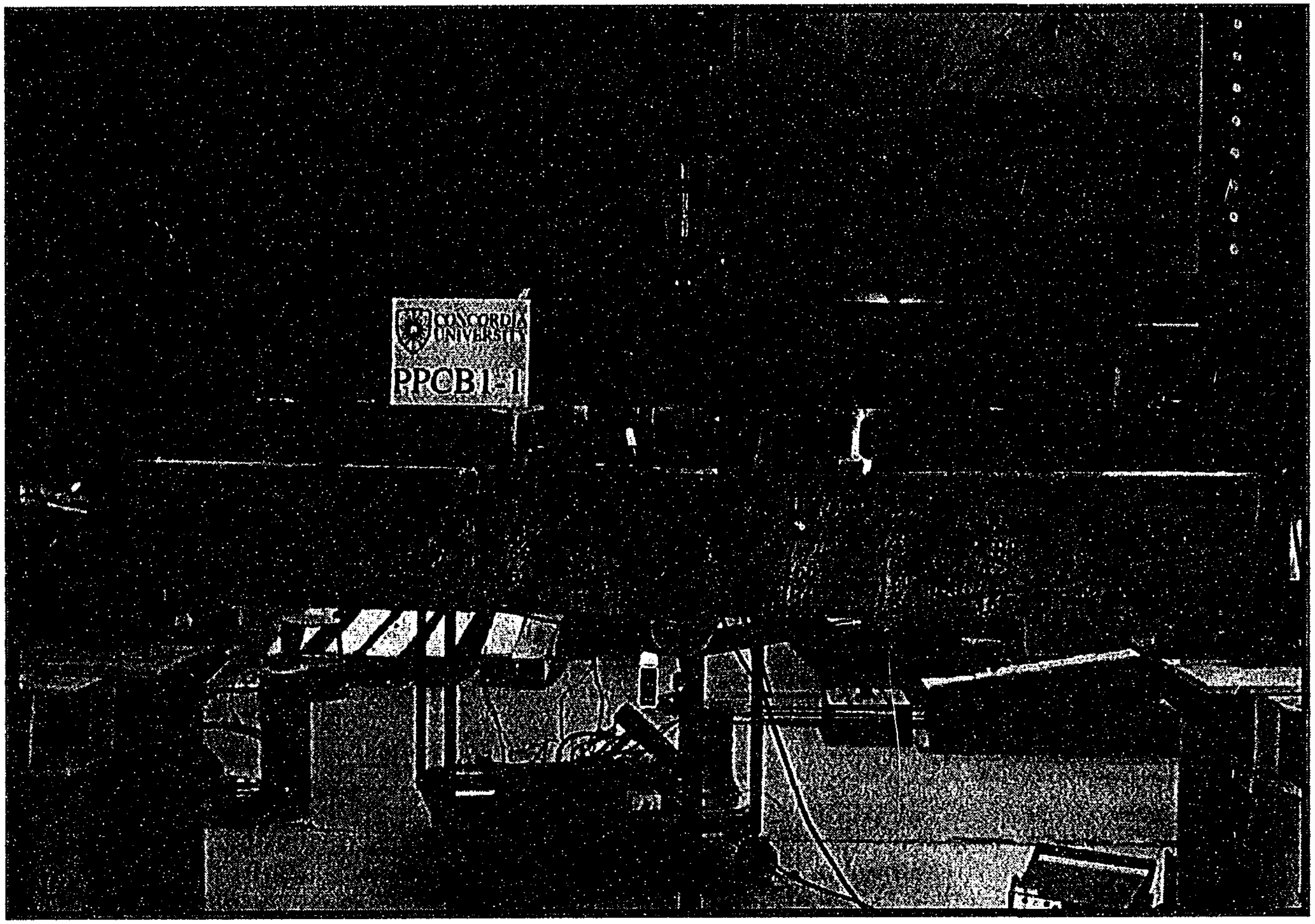


Figure A.5 Beam PPCB1-1 at Failure

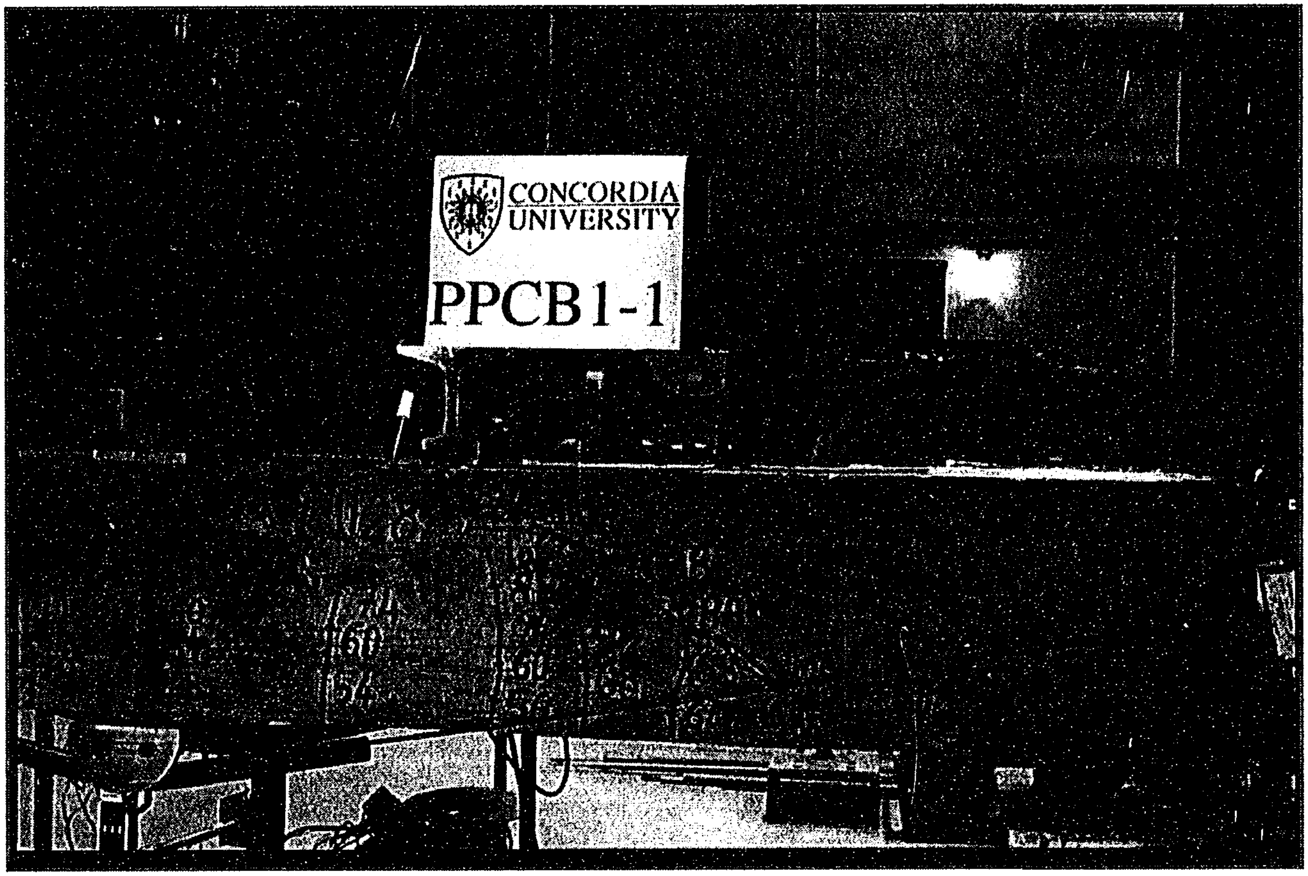


Figure A.6 Close-up of Failure of Beam PPCB1-1

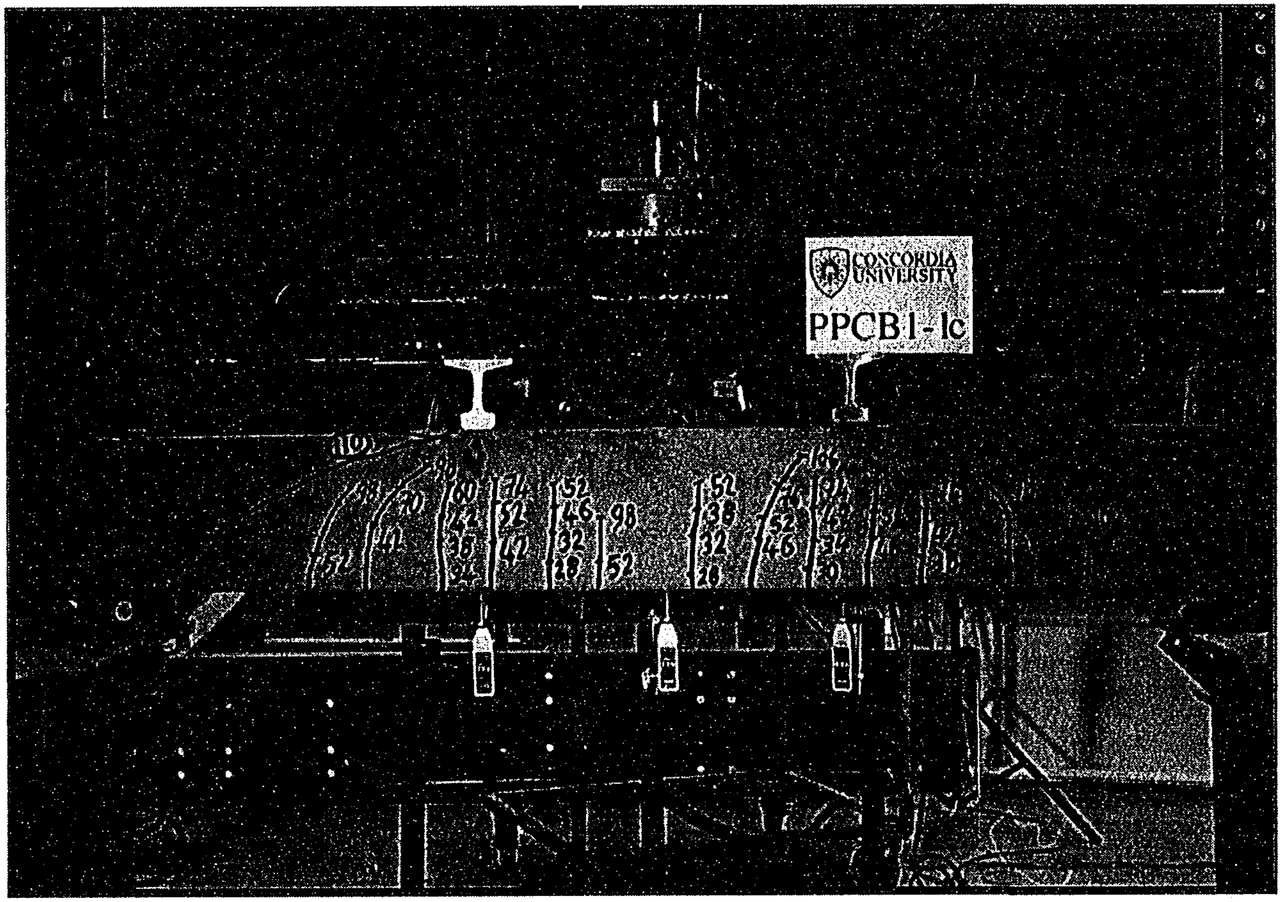


Figure A.7 Control Beam PPCB1-1c at Failure

Table A.3: Behaviour of Beam PPCB1-2 during Testing

L= 2.00 m S = 2.25 m S/d_p = 10.7 b = 150 mm h = 280 mm d_s = 250 mm d_{ps} = 210 mm d_{pe} = 324 mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) f_y = 400 MPa E_s = 200000 MPa A_s' = 2-6mm plain bars (56.62mm^2) f_y' = 275 MPa	Internal Prestressing: A_{ps} = 2-3/8 in strands (110.0 mm^2) f_{pu} = 1860 MPa $f_{py} = 0.9 f_{pu}$ (low relaxation) $f_{pe} = 0.6 f_{pu}$ E_{ps} = 200000 MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 7.5mm A_{pf} = 2(30.4) = 60.8mm^2 f_{puf} = 2040 MPa $f_{pef} = 0.5 f_{pu}$ E_{pf} = 135000 MPa	f_c' = 35 MPa (Design) f_c' =47.83 MPa (at time of testing) Moment at midspan = PL/6

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	30	1.8	79.4	190	270	0.08	4	----	First crack (hair line crack).
	40	2.73	174.6	287	200	0.08	5	----	Flexural cracks developed.
	48	3.65	231	348	147	0.15	8	----	New crack formed in the region of constant moment and shear span
	54	4.33	278.4	396	136	0.25	10	----	Maximum crack depth is half the beam depth.
No time dependent effects. Beam was strengthened directly.									

Table A.3: Behaviour of Beam PPCB1-2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
external prestressing	55	4.05	259.4	356	136	0.25	10	10	
	56	3.61	220	311	136	0.22	10	20	
	59	3.11	168.6	254	136	0.08	10	35	Most cracks closed.

Table A.3: Behaviour of Beam PPCB1-2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Loading after strengthening	62	4.49	173.6	275	136	0.08	10	35	
	76	5.12	204.8	348	136	0.08	10	35	
	84	5.54	230.2	393	122	0.1	11	35	New crack formed in the shear span and some cracks started to reopen.
	116	7.75	356.2	603	122	0.3	11	36	Cracks propagated upward.
	136	9.78	442	748	108	0.4	13	37	Cracks propagated towards the point load.
	148	11.28	475.8	849	109	0.5	14	38	
	160	15.01	897	1039	114	0.8	17	39	Cable broke and beam did not fail. Cracks were stabilized at maximum depth equal to 2/3 the beam depth.
	106	14.87	846.8	1027	114	0.8	17	0	Load dropped after breakage of cable.
	110	15.04	857.2	1043	114	0.8	17	0	The applied load was increased to cause failure in the beam itself.

Table A.3: Behaviour of Beam PPCB1-2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in Reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Loading after strengthening	134	19.13	1013.2	1292	114	0.9	17	0	The shear cracks propagated towards the point load.
	142	21.28	1114.6	1417	114	1.0	17	0	New cracks initiated at load P=160kN and propagated towards the point load. Slip cracks formed near the supports. Beam failed in Shear.
	154	25.92	n/a	1541	114	>1.5	17	0	
releasing the applied load.	0	11.1	n/a	491	114	1.5	17	0	

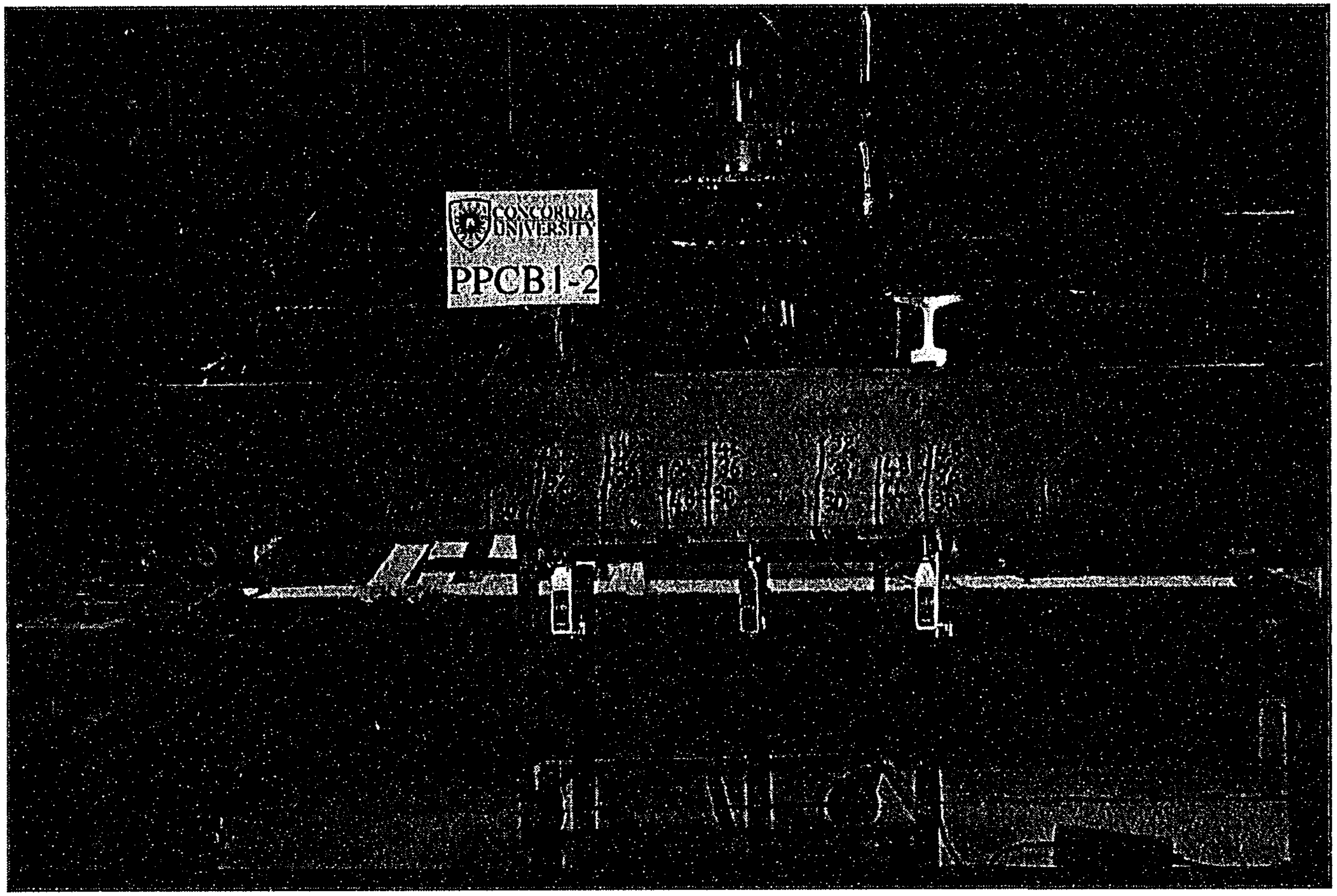


Figure A.8 Beam PPCB1-2 after Application of Initial Damaging Load

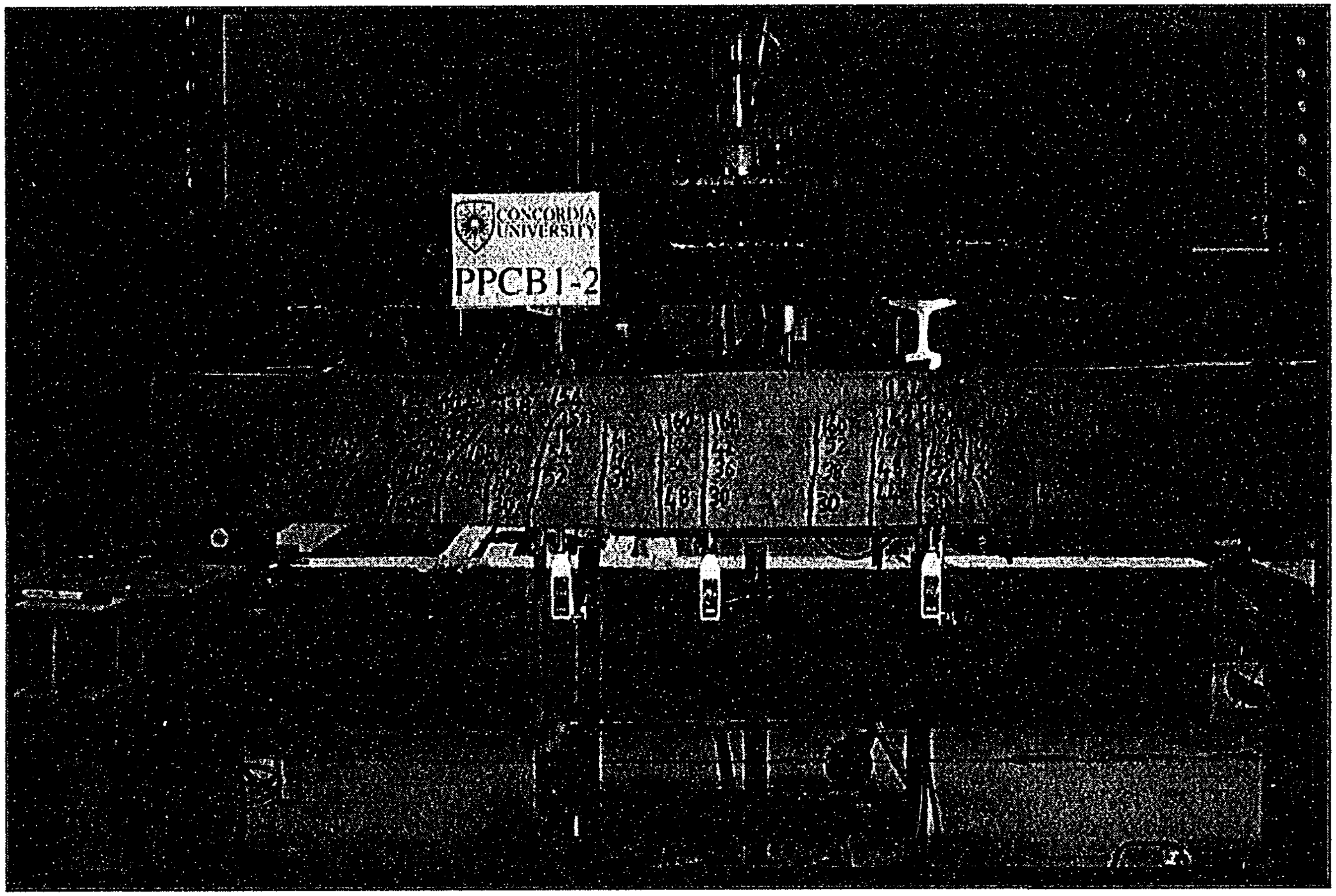


Figure A.9 Beam PPCB1-2 at Failure

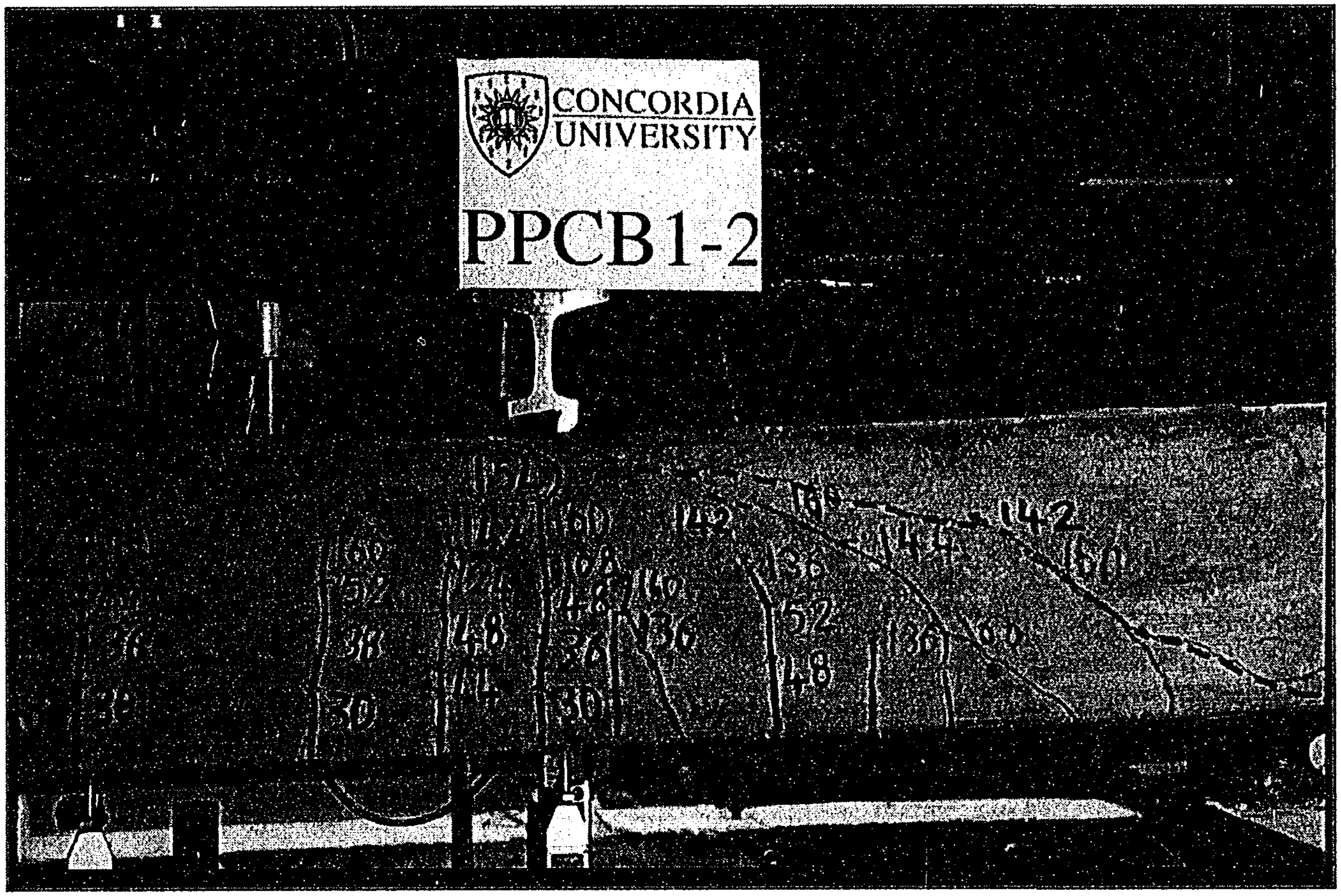


Figure A.10 Close-up of Failure of Beam PPCB1-2

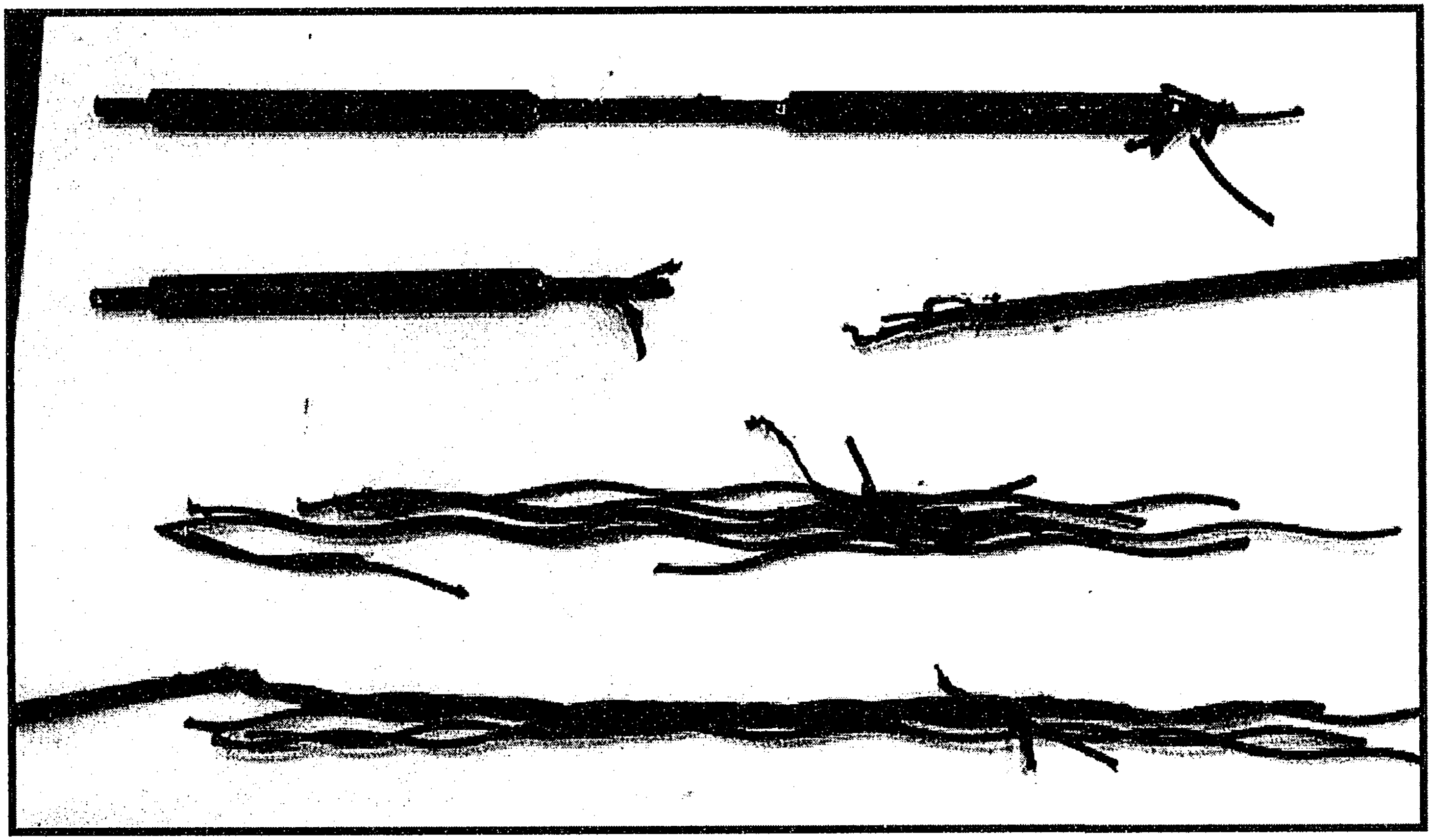
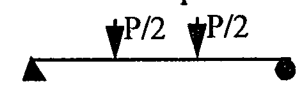


Figure A.11 Cable after Breakage

Table A.4: Behaviour of Beam RCB2 during Testing

L = 3.50 m S = 3.75m S/d_p = 17.8 b = 150 mm h = 280 mm d_s = 250 mm d_{ps} = 210 mm d_{pe} = 324 mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) f_y = 400 MPa E_s = 200000 MPa A_s' = 2-6mm plain bars (56.62mm^2) f_y' = 275 MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 5.0mm A_{pf} = 2(10.1) = 20.2mm^2 f_{puf} = 2120 MPa f_{pef} = 0.5 f_{pu} E_{pf} = 144000 MPa	f_c' = 35 MPa (Design) f_c' = 42.41 MPa (at time of testi Moment at midspan = PL/6 

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	10	2.02	45	100	----	----	1	-----	First crack (hair line crack).
	18	7.60	339.6	318	200	----	9	-----	Max. damaging load.
	0	2.56	n/a	71	200	----	9	-----	Applied load released. Steel strain gauge were damaged.
sustained up to 82 days	1	1.7	n/a	253	200	----	9	-----	Own weight only.
reloading before external prestressing	0	0	n/a	0	200	----	9	-----	
	8	2.3		118	192	0.08	14	-----	New cracks formed in the region of constant moment and in the shear span.
	18	6.26		293	192	0.15	14	-----	Crack propagated to 2/3 beam depth

Table A.4: Behaviour of Beam RCB2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
external prestressing	18	6.26	n/a	302	192	0.1	14	0	
	18	6.21		294	192	0.1	14	10	
loading after strengthening	28	15.31	n/a	462	192	0.15	14	15	Values of deflection, strain and stress include the residuals from the initial damaging load and time dependent effects.
	30	16.38		496	130	0.2	18	16	new cracks formed.
	36	20.82		636	130	0.33	18	21	
	42	34.71		1054	130	1.0	21	40	cracks stabilized.
	48	64.52		1472	122	>1.5	22	74	Crushing at top started at the point load with longitudinal crack directed inside the constant moment region. Slip crack occurred.
	52	118.96		1295	122	n/a	22	122	Flexural failure with longitudinal crack at top under the point load.
releasing applied load	45	110.07	n/a	1295	122	n/a	22	122	
	0	104.11		545	122	n/a	22	97	

Table A.4: Behaviour of Beam RCB2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
releasing external prestressing	0	103.54	n/a	560	122	n/a	22	74	
	0	94.73		452	122	n/a	22	69	
	0	54.81		422	122	n/a	22	63	an upward load was applied from bottom to release the external cable

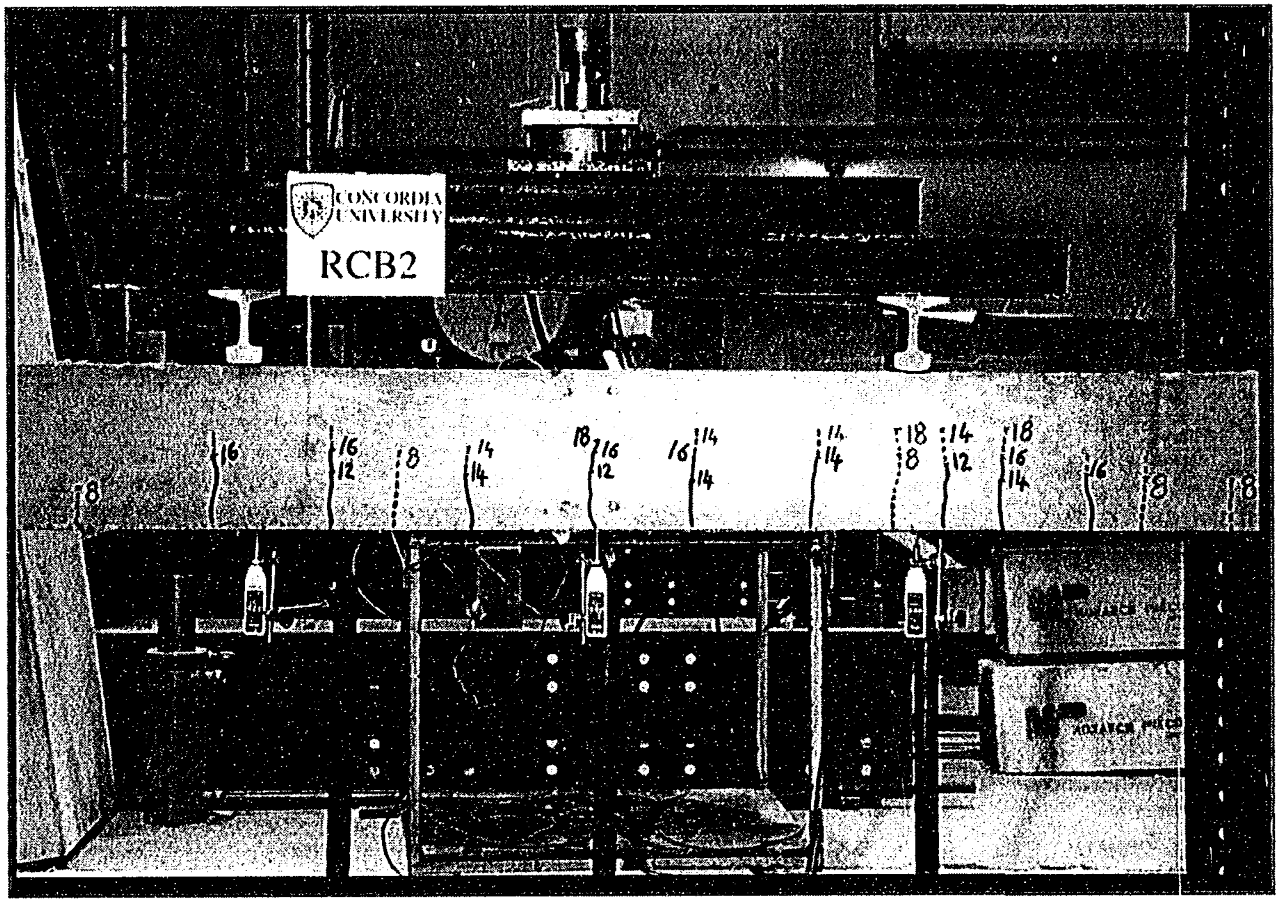


Figure A.12 Beam RCB2 just before Application of External Prestressing

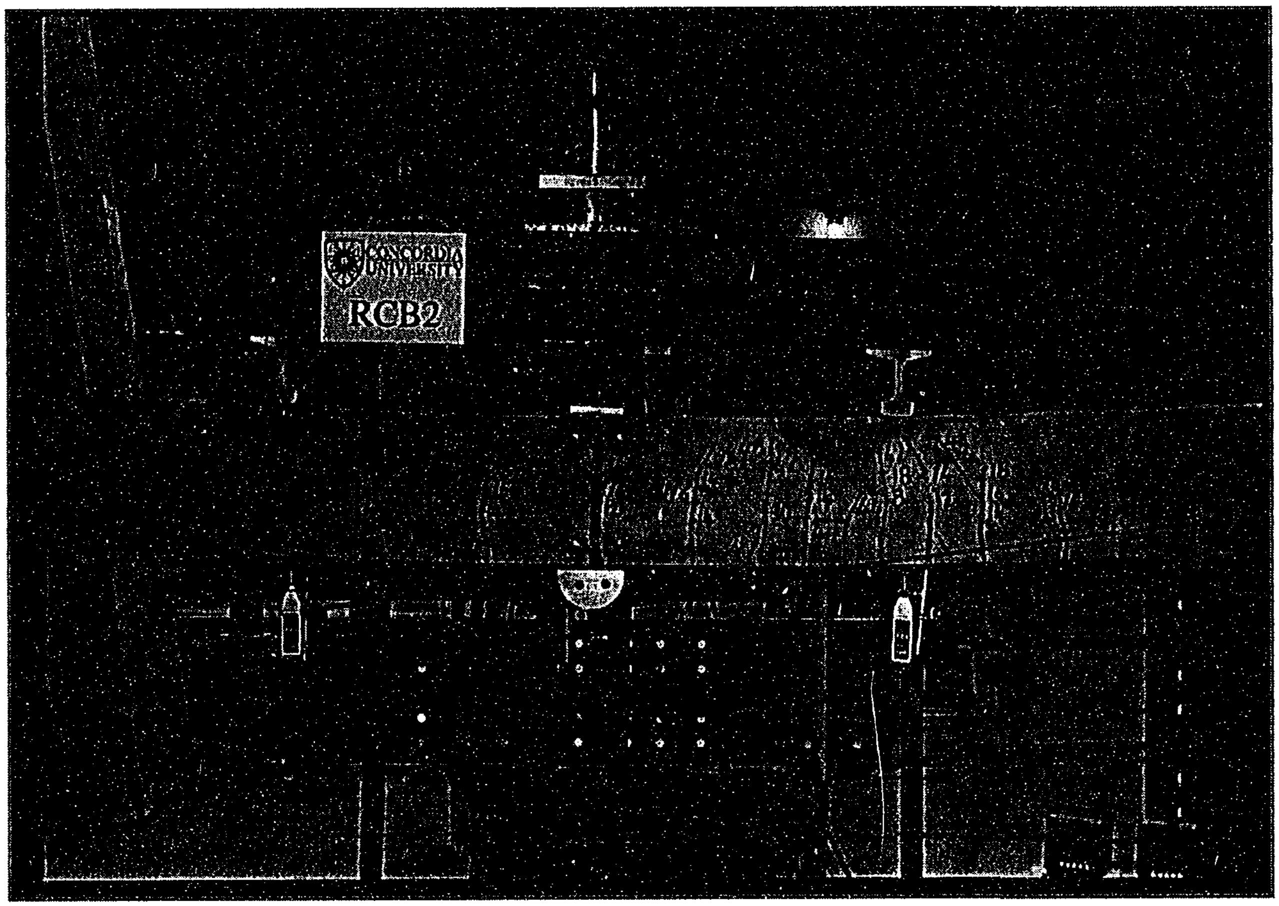


Figure A.13 Beam RCB2 at Failure

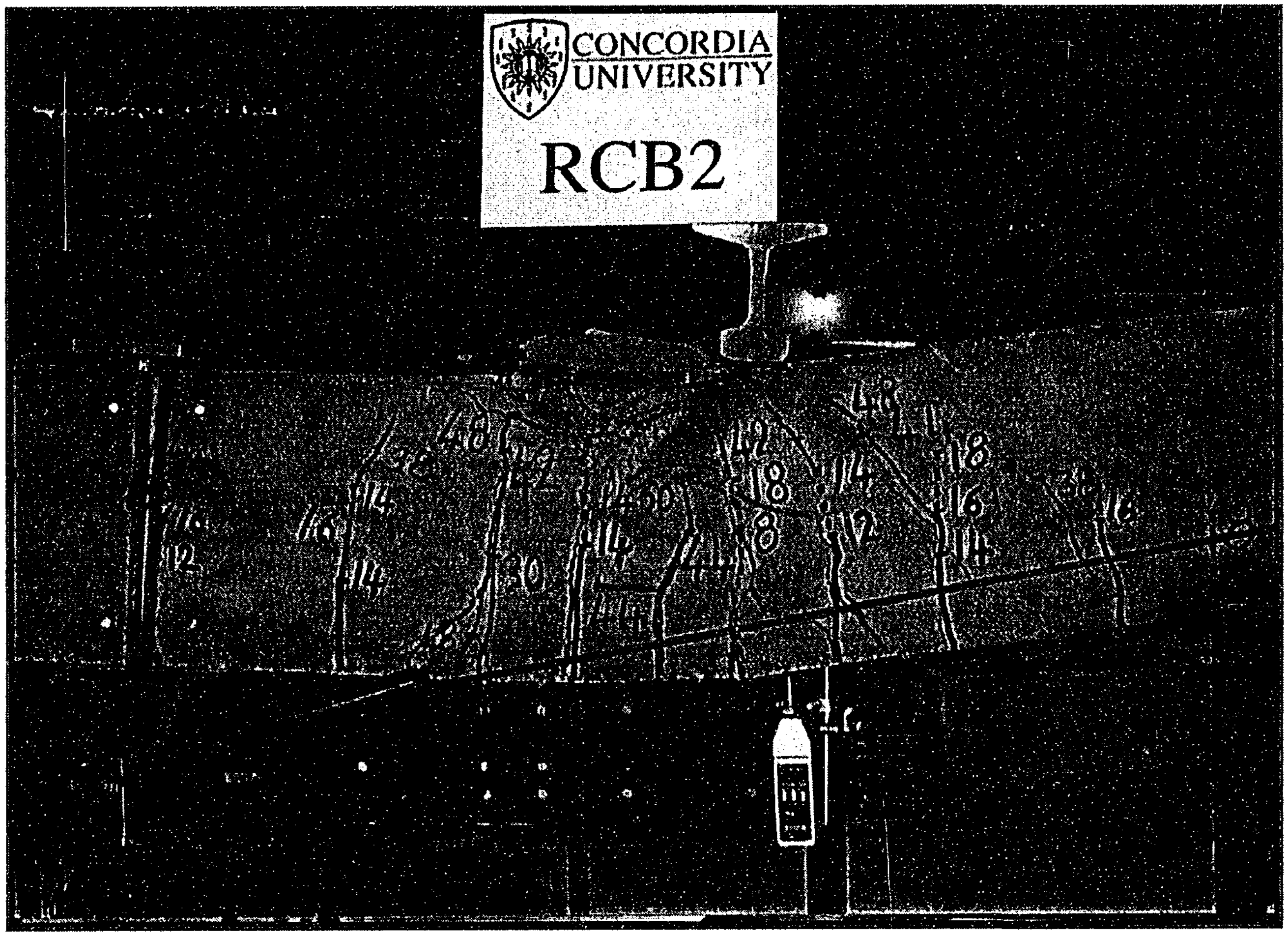


Figure A.14 Close-up of Failure of Beam RCB2

Table A.5: Behaviour of Beam PPCB2-1 during Testing

L = 3.50 m S = 3.75m S/d_p = 17.8 b = 150 mm h = 280 mm d_s = 250 mm d_{ps} = 210 mm d_{pe} = 324 mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) f_y = 400 MPa E_s = 200000 MPa A_s' = 2-6mm plain bars (56.62mm^2) f_y' = 275 MPa	Internal Prestressing: A_{ps} = 1-3/8 in strands (55.0 mm^2) f_{pu} = 1860 MPa $f_{py} = 0.9 f_{pu}$ (low relaxation) $f_{pe} = 0.6 f_{pu}$ E_{ps} = 200000 MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 5.0mm A_{pf} = 2(10.1) = 20.2mm^2 f_{puf} = 2120 MPa $f_{pef} = 0.5 f_{pu}$ E_{pf} = 144000 MPa	f_c' = 35 MPa (Design) f_c' = 44.2 MPa (at time of testing) Moment at midspan M = PL/6

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	14	2.75	n/a	180	320	hair line	4	-----	Strain gauges attached to the steel were damaged during casting.
	18	4.72		310	235	0.08	8	-----	Cracks formed in the region of constant moment, and max. crack depth was 2/3 the beam depth.
	24	9.35		468	153	0.2	13	-----	
No time dependent effects. Beam was strengthened directly.									

Table A.5: Behaviour of Beam PPCB2-1 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
external prestressing	24	9.35	n/a	487	153	0.2	13	0	Initial cable length = 3715mm
	24	9.1		466	153	0.15	13	10	Camber equal to 0.25 due to external prestressing.
loading after strengthening	26	9.52	n/a	497	153	n/a	13	10	
	28	10.48		529	125	n/a	16	11	New cracks formed in the shear span close to the point load.
	30	11.09		562	115	0.2	16	12	All cracks propagated equally at the same load levels.Almost symmetric cracking pattern under the two point loads. At load P= 50kN the flexural cracks started to divert towards the point load.
	40	16.32		754	114	0.3	20	17	
	62	44.96		1564	110	1.0	24	47	
	66	52.21		1764	110	1.25	25	51	
	80	101.74		2425	110	>1.5	25	104	Flexural shear failure with longitudinal crack at top close to the point load, directed towards the constant moment region and exceeded the midspan of the beam. Cable length at failure =3750mm

Table A.5: Behaviour of Beam PPCB2-1 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
releasing applied load	62	101.74	n/a	1109	110		25	104	
	0	92.2		810	110		25	58	
releasing external prestressing	0	91.2	n/a	-----	110		25	25	
	0	90.89		-----	110		25	0	Cable length after releasing the external prestressing force was equal to 3718mm

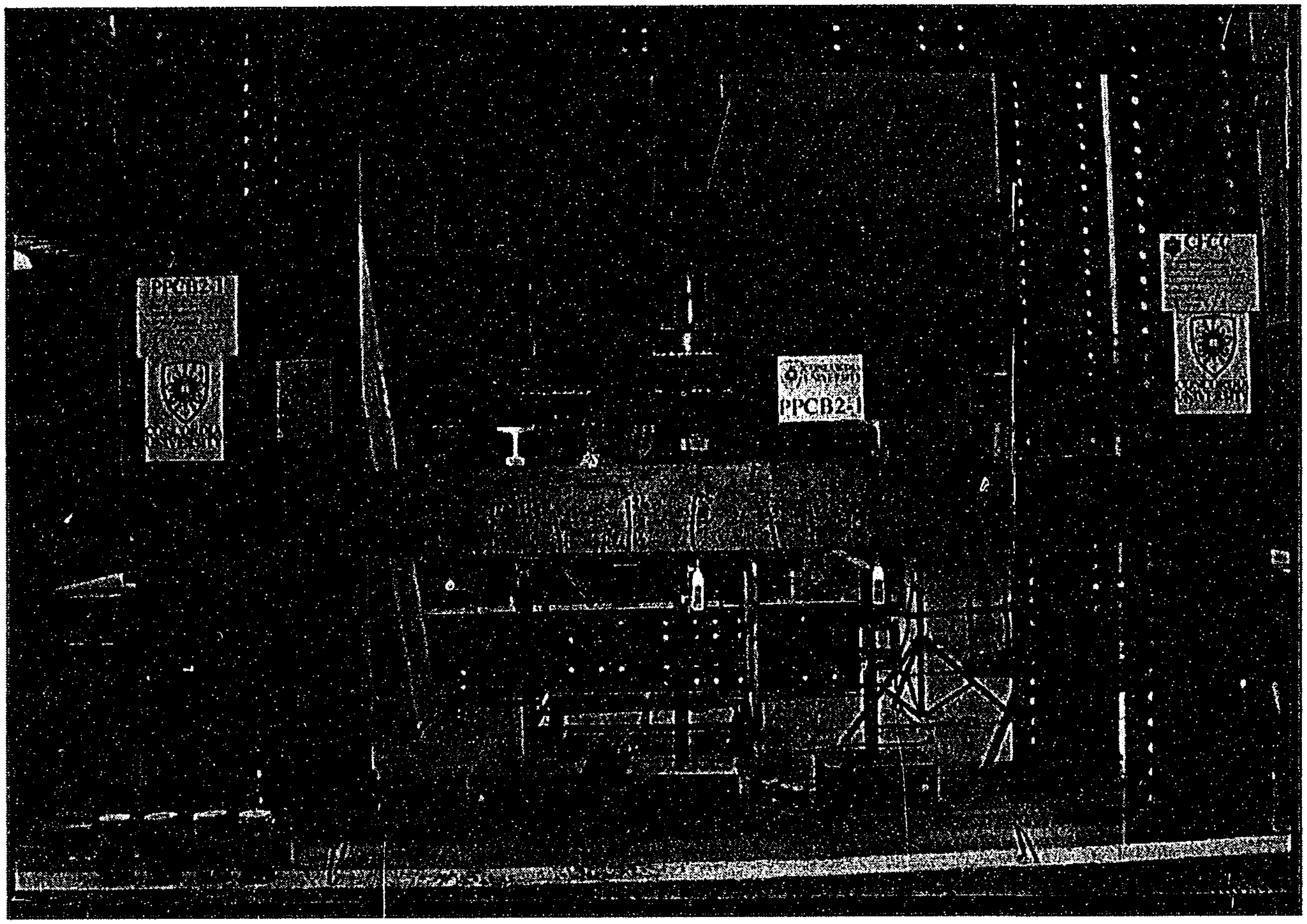


Figure A.15 Beam PPCB2-1 after Application of Initial Damaging Load

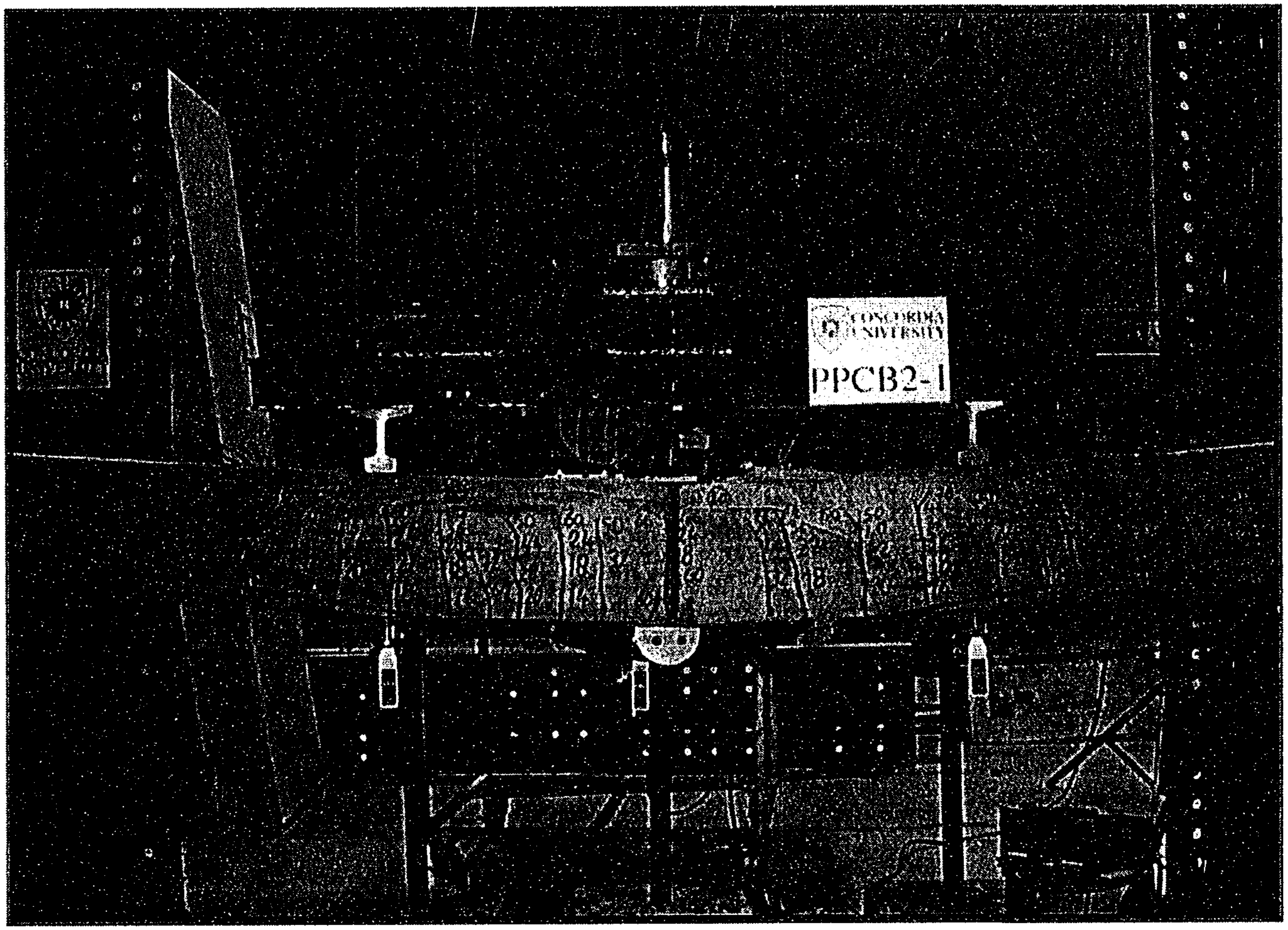


Figure A.16 Beam PPCB2-1 at Failure

Figure A.17 Close-up of Failure of Beam PPCB2-1

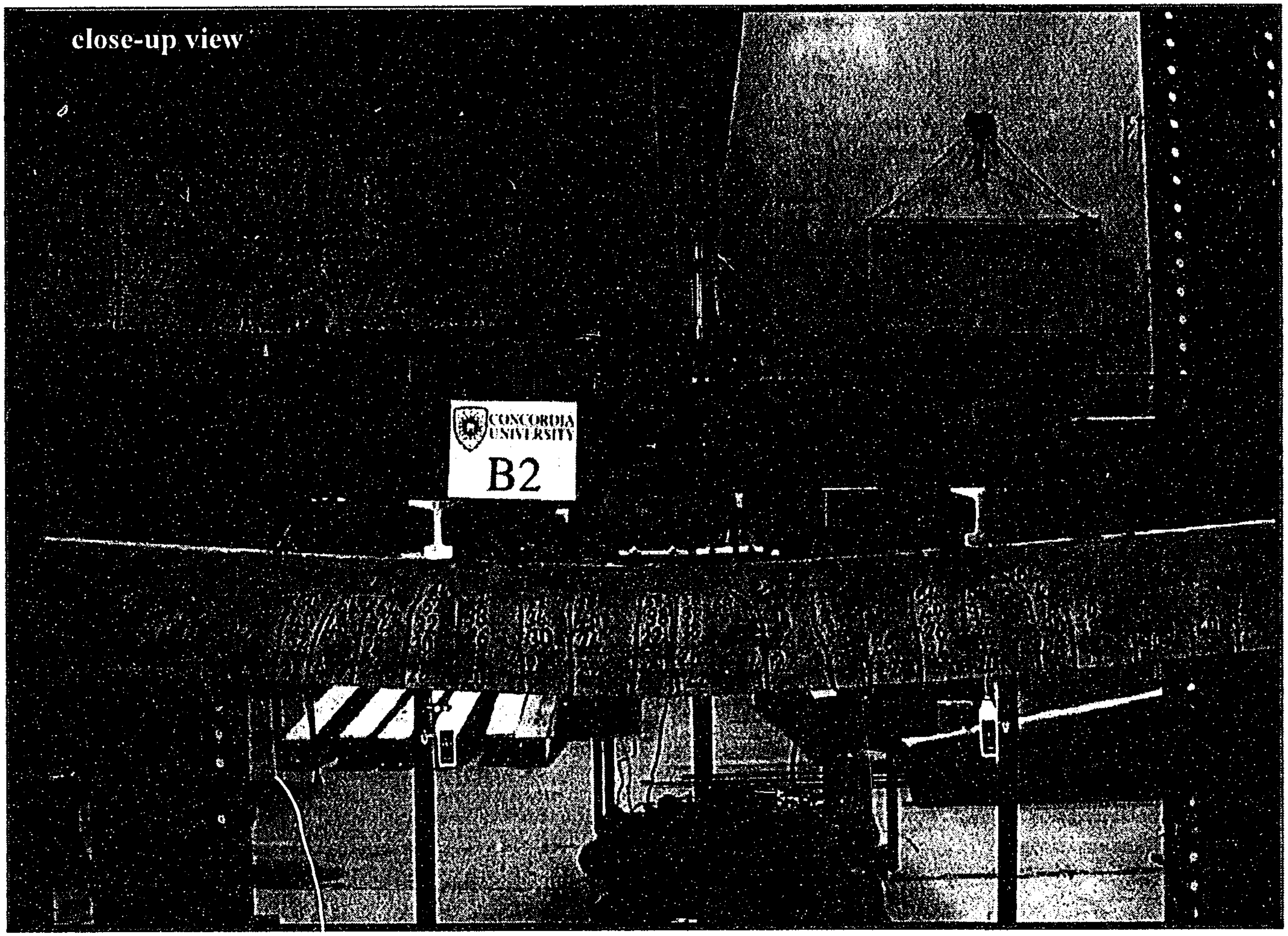


Figure A.18 Control Beam PPCB2-1c at Failure

Table A.6: Behaviour of Beam PPCB2-2 during Testing

$L = 3.50$ m $S = 3.75$m $S/d_p = 17.8$ $b = 150$ mm $h = 280$ mm $d_s = 250$ mm $d_{ps} = 210$ mm $d_{pe} = 324$ mm	Ordinary Reinforcement: $A_s = 2\#10M$ (200 mm^2) $f_y = 400$ MPa $E_s = 200000$ MPa $A_s' = 2$-6mm plain bars (56.62mm^2) $f_y' = 275$ MPa	Internal Prestressing: $A_{ps} = 2$-3/8 in strands (110.0 mm^2) $f_{pu} = 1860$ MPa $f_{py} = 0.9\ f_{pu}$ (low relaxation) $f_{pe} = 0.6 f_{pu}$ $E_{ps} = 200000$ MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 7.5mm $A_{pf} = 2(30.4) = 60.8mm^2$ $f_{puf} = 2.04\ kN/mm^2$ $f_{pef} = 0.5 f_{pu}$ $E_{pf} = 144000$ MPa	$f_c' = 35$ MPa (Design) $f_c' = 49.16$ MPa (at time of testing) Moment at midspan M = PL/6

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	16	3.23	131.6	201	180	----	6	-----	First crack (hair line crack)
	18	4.03	163.8	236	175	n/a	7	-----	New cracks formed in the region of constant moment
	22	6.03	252.2	322	177	0.08	9	-----	Cracks formed in both the constant moment region and the shear span near the point load.
	24	7.11	270.8	376	183	n/a	11	-----	New cracks formed in the shear span near the point load.
	26	8.35	302.4	415	165	n/a	13	-----	New cracks under the point load. Max. crack depth was 1/2 th depth of beam.

Table A.6: Behaviour of Beam PPCB2-2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
No time dependent effects. Beam was strengthened directly.									
external prestressing	26	8.35	306	424	165	n/a	13	0	
	27	7.8	263	388	165	n/a	13	10	
	26	4.94	135.2	242	165	n/a	13	22	Most cracks closed.
	29	4.32	98.8	210	165	n/a	13	30	Camber = 4.03 mm.
loading after strengthening	27	4.32	92	195	165	n/a	13	30	
	36	6.11	135.4	299	165	n/a	13	30	Cracks started to reopen.
	44	7.98	197.2	399	152	n/a	14	31	New cracks formed in the shear span near the point loads.
	56	7.18	305.8	569	124	0.1	17	32	
	64	14.78	392.2	702	116	0.15	18	33	All cracks propagated equally. Almost symmetric cracking pattern was observed over the entire span.
	70	17.37	463.4	805	116	0.2	19	34	
	80	21.52	580.8	965	114	0.25	22	35	Cracks in the shear span near the point load started to divert towards the point loads.
	92	30.26	692.4	1226	113	0.6	24	37	Cracks stabilized.

Table A.6: Behaviour of Beam PPCB2-2 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
loading after strengthening	102	37.57	n/a	1460	113	0.8	24	40	Max. crack depth was 2/3 the beam depth
	106	42.37		1612	113	0.8	24	40	Cable broke, but beam did not fail.
	72	33.18		n/a	113	0.8	24	0	The applied load dropped dawn.
releasing the applied load	0	11.94	n/a	n/a	113	0.8	24	0	
reloading up to failure	0	11.94	n/a	472	113	n/a	24	-----	The beam was retedted after 67 days. The load was reapplied to cause failure in the beam itself. Beam failed in Shear
	96	74.08		2864				-----	
	72	73.79		2722				-----	The applied load dropped dawn.
releasing the applied load	0	33.15		1019				-----	

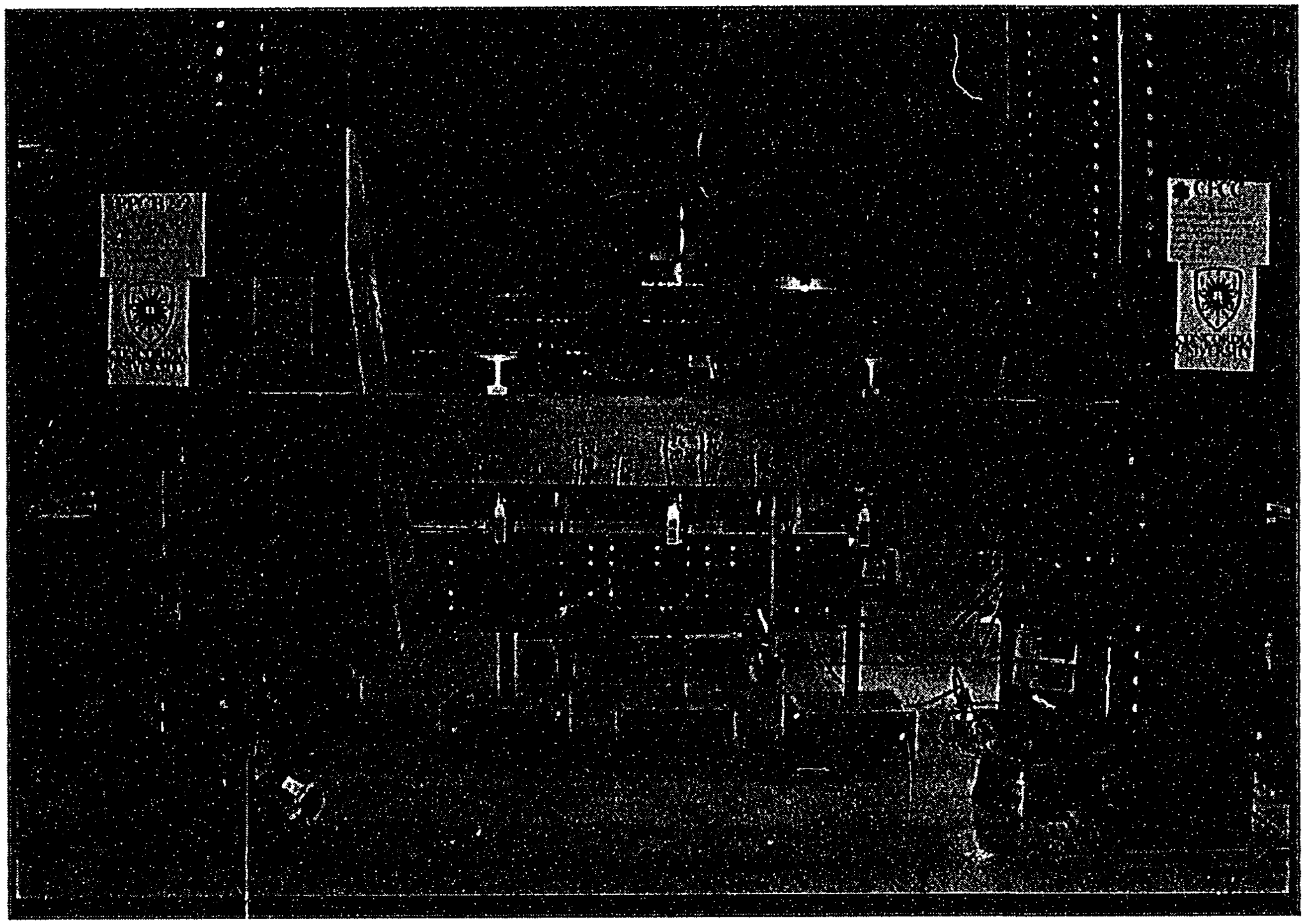


Figure A.19 Beam PPCB2-2 after Application of Initial Damaging Load

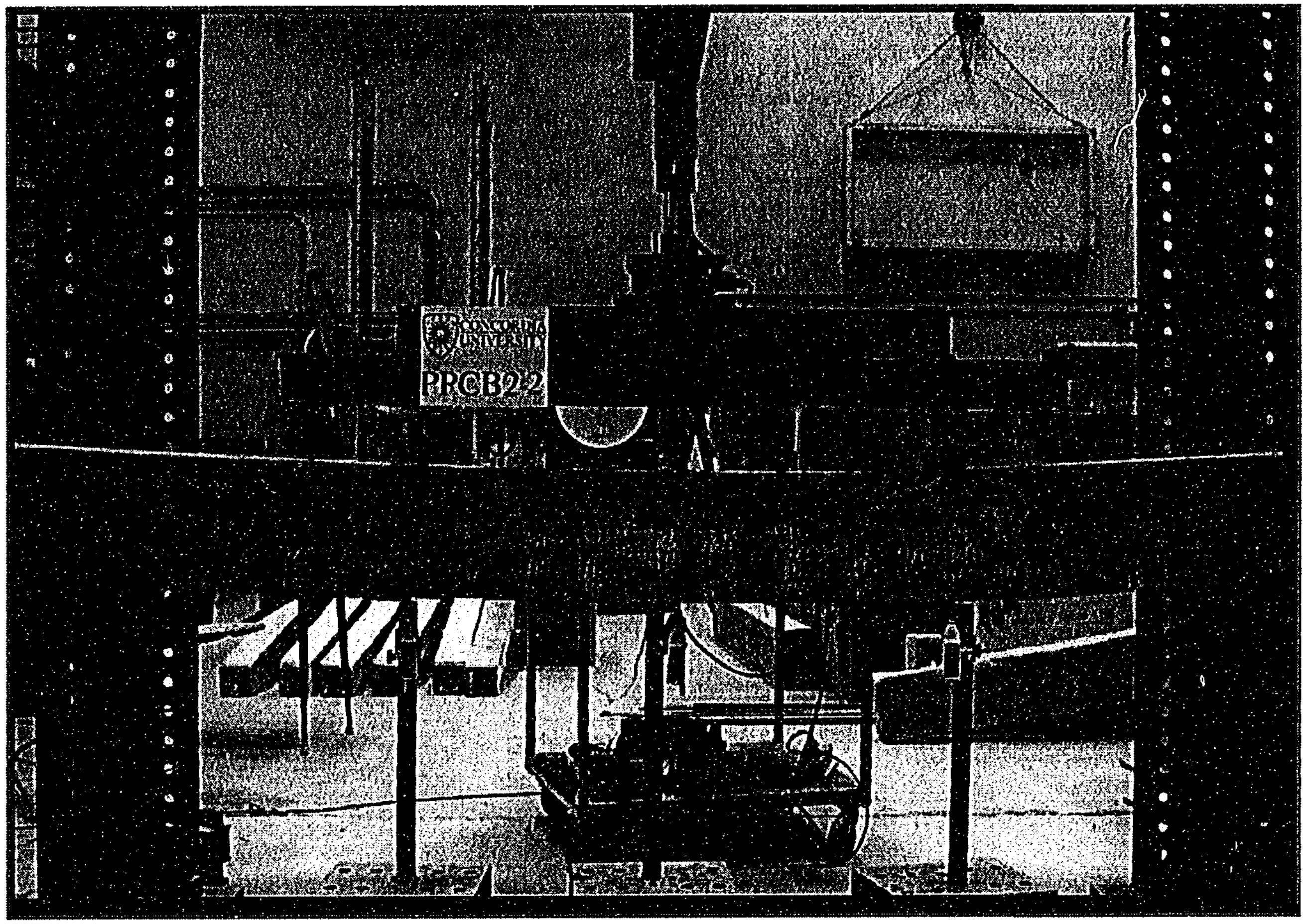


Figure A.20 Beam PPCB2-2 at Failure

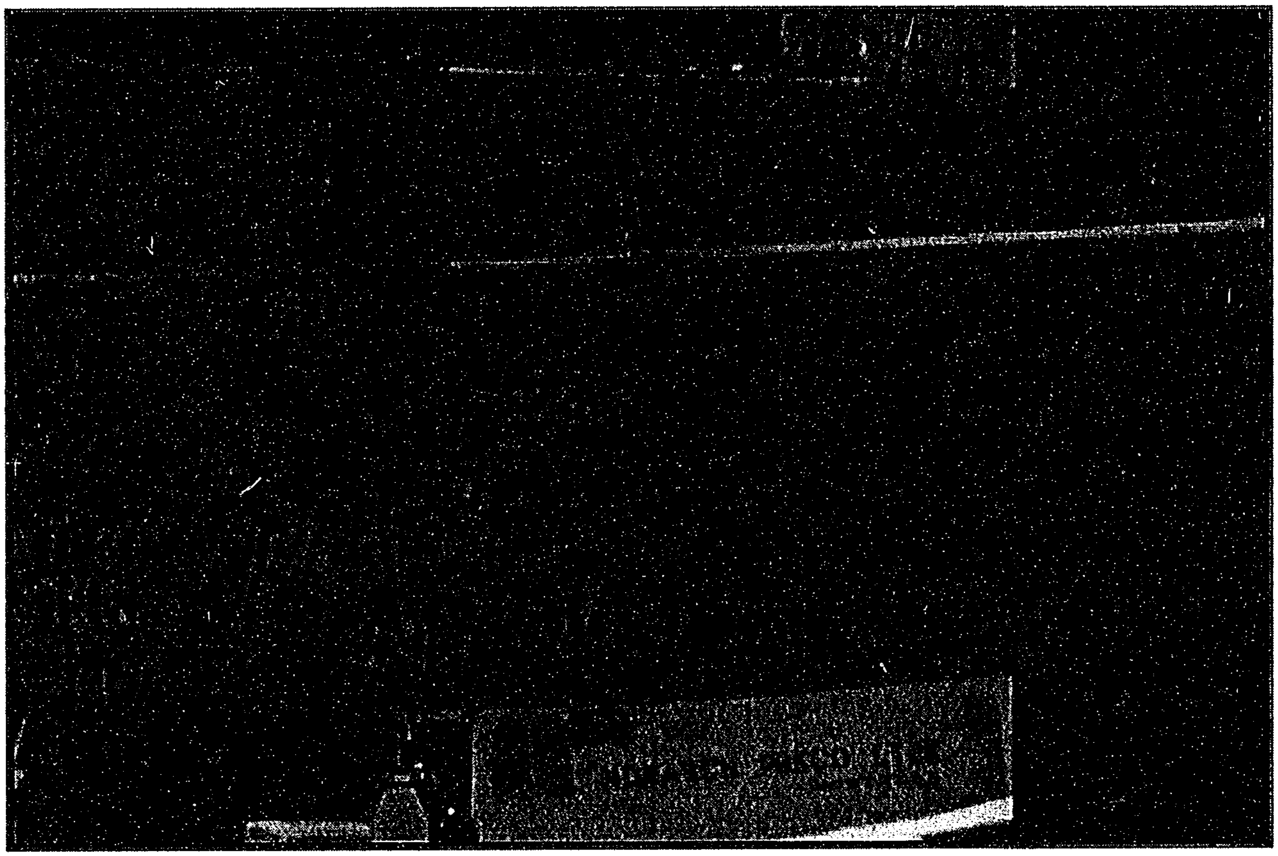
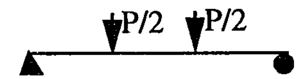


Figure A.21 Close-up of Failure of Beam PPCB2-2

Table A.7: Behaviour of Beam RCB3 during Testing

L = 5.00 m S = 5.25 m $S/d_p = 25$ b = 150 mm h = 280 mm d_s = 250 mm d_{ps} = 210 mm d_{pe} = 324 mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) f_y = 400 MPa E_s = 200000 MPa A_s' = 2-6mm plain bars (56.62mm^2) f_y' = 275 MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 5.0mm $A_{pf} = 2(10.1) = 20.2\text{mm}^2$ $f_{puf} = 2.12\ \text{kN/mm}^2$ $f_{pef} = 0.5 f_{pu}$ E_{pf} = 144000 MPa	f_c' = 35 MPa (Design) f_c' = 36.2 MPa (at time of testing) Moment at midspan = PL/6



Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	4	1.34	35.6	57	----	0.08	1	-----	First crack.
	12	12.56	216.2	369	185	0.33	14	-----	Max. damaging load. Max. crack propagated to 1/2 the beam depth.
	0	4.78	57.8	107	185	0.33	14	-----	Residual deflection, strain, stress.
sustained up to 82 days	1	3.65	163.8	324	185	0.33	14	-----	Own weight only
reloading before external prestressing	0	8.43	186.4	431	185	0.08	14	-----	Total residual deflection values are (4.78+3.65)
	12	14.81	220.8	1175	185	0.33	14	-----	Max. crack propagated to 2/3 the beam depth

Table A.7: Behaviour of Beam RCB3 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
external prestressing	12	14.81	220.8	1200	185	0.33	14	0	
	12	15.69	227.2	1227	185	0.3	14	10	
loading after strengthening	22	24.22	305.4	1417	180	0.33	18	14	New cracks formed in the shear span near the point load, and in the middle. Slip crack occurred under the point load. Crack propagated to a max. depth of 3/4 the beam depth.
	26	27.76	291.6	1524	170	0.4	21	17	Cracks stabilized.
	39	123.1	334.4	2626	170	0.6	21	88	Flexural failure under the point load. A longitudinal crack formed under the right point load propagated inside and part by outside the region of constant moment.
unloading	0	n/a	n/a	n/a	170	0.6	21	62	The beam was unloaded to add extension to the jack, and then reloaded. Cracks were very wide.
reloading	42	n/a	n/a	n/a	170	>>>	21	143	
releasing the applied load	0	n/a1	n/a6	n/a5	170	>>>	21	109	
releasing external prestressing force: an upward load was applied from bottom to release the external cable									

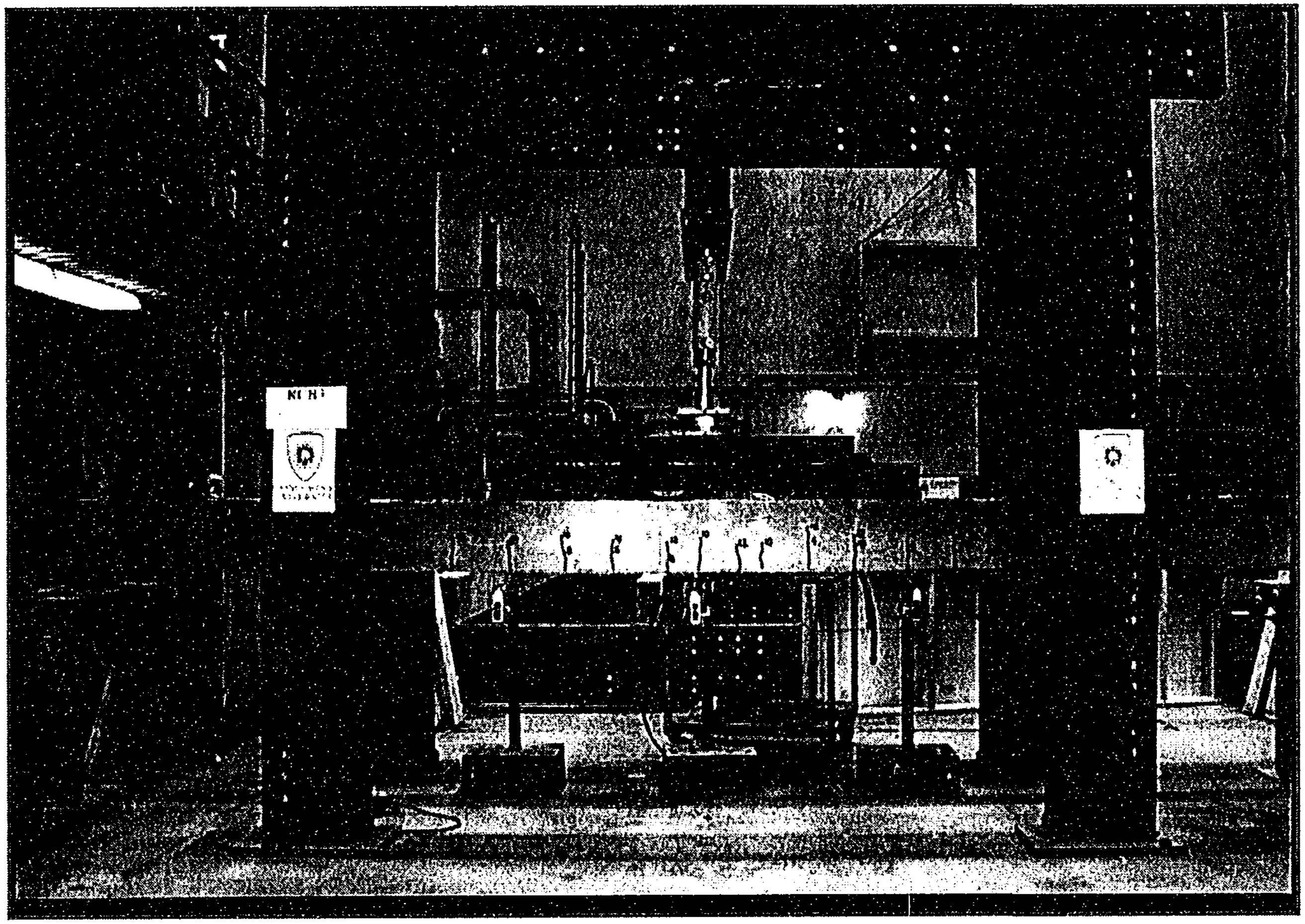


Figure A.22 Beam RCB3 after Application of Initial Damaging Load

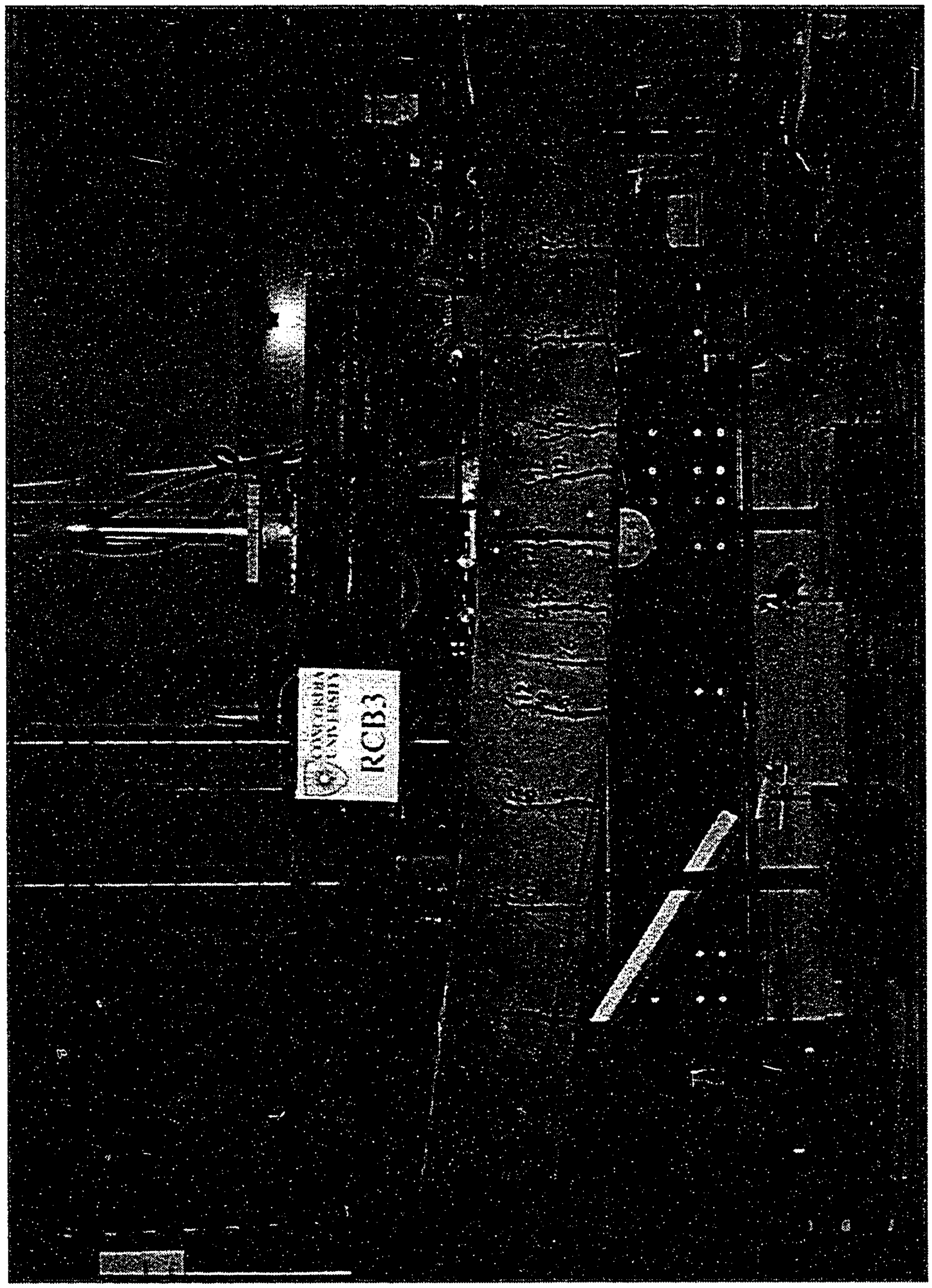


Figure A.23 Beam RCB3 at Failure

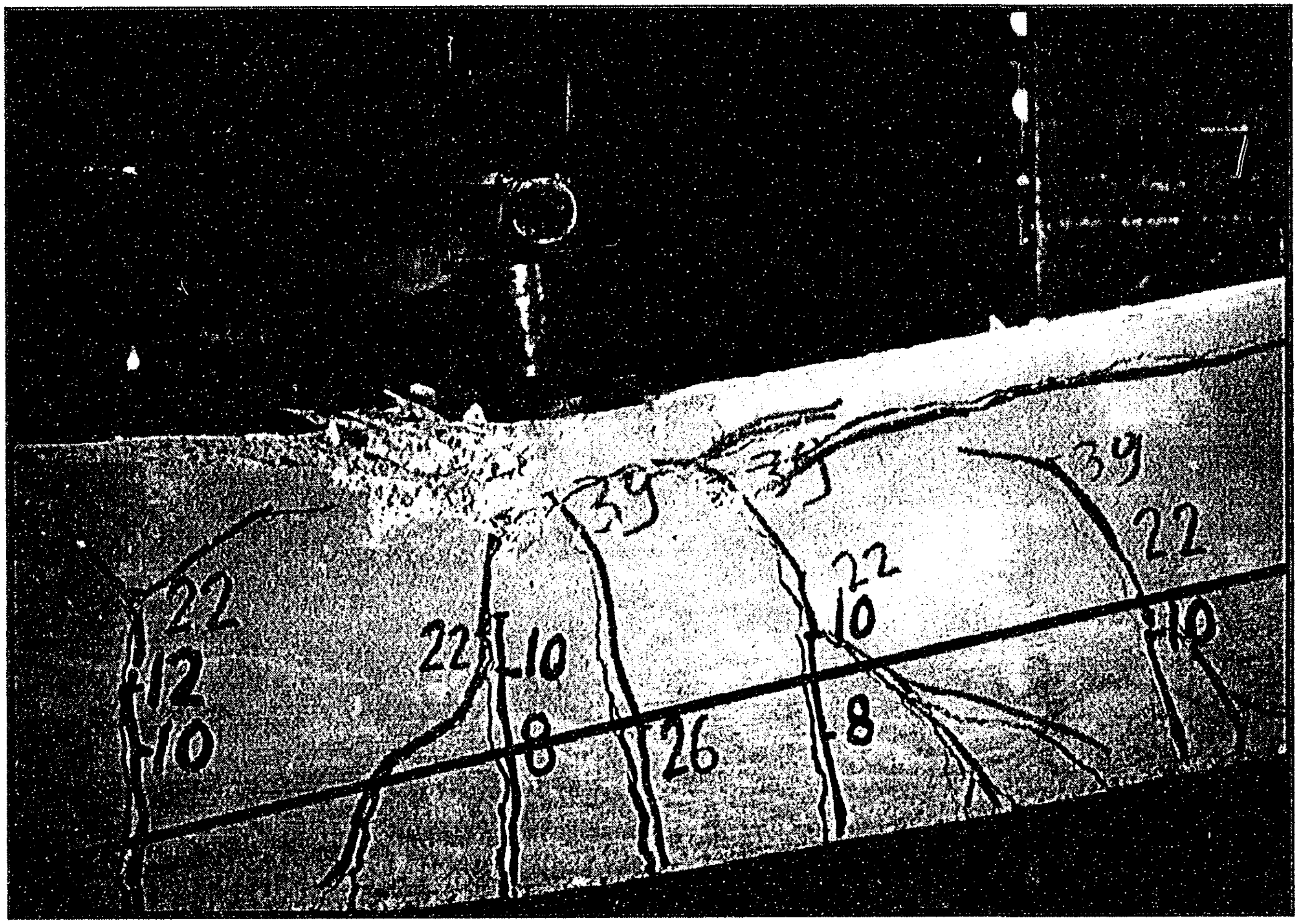


Figure A.24 Close-up Failure of Beam RCB3

Table A.8: Behaviour of Beam PPCB3-1 during Testing

L =5.00 m S = 5.25 m $S/d_p = 25$ b = 150 mm h = 280 mm $d_s = 250$ mm $d_{ps} = 210$ mm $d_{pe} = 324$ mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) $f_y = 400$ MPa $E_s = 200000$ MPa A_s' = 2-6mm plain bars (56.62mm^2) $f_y' = 275$ MPa	Internal Prestressing: A_{ps} = 1-3/8 in strands (55.0 mm^2) $f_{pu} = 1860$ MPa $f_{py} = 0.9 f_{pu}$ (low relaxation) $f_{pe} = 0.6 f_{pu}$ $E_{ps} = 200000$ MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 5.0mm $A_{pf} = 2(10.1) = 20.2\text{mm}^2$ $f_{puf} = 2120$ MPa $f_{pef} = 0.5 f_{pu}$ $E_{pf} = 144000$ MPa	$f_c' = 35$ MPa (Design) $f_c' = 49.15$ MPa (at time of testing) Moment at midspan M = PL/6

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	20	4.48	40.2	166	200	----	4	----	First cracks.
	30	14.6	217	420	157	----	14	----	Cracks formed in the region of constant moment, and max. crack depth was half the beam depth.
	0	2.59	41.2	44	157	----	14	----	
sustained up to 81 days	1	0.92	143	97	157	----	14	----	Own weight only.
reloading before external prestressing	0	3.51	184.2	141	157	----	14	----	With residual values.
	14	10.92	243	375	157	----	14	----	Cracks started to reopen.
	24	18.95	285.4	571	156	0.1	17	----	New cracks formed.
	30	26.43	305.4	674	127	0.3	22	----	Cracks propagated to 2/3 the beam depth

Table A.8: Behaviour of Beam PPCB3-1 during Testing (cont'd)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
external prestressing	30	26.43	305.4	706	127	0.3	22	0	
	30	26.43	300	697	127	0.3	22	10	
loading after strengthening	36	32.21	325.4	800	127	0.33	22	13	
	38	34.83	343.4	840	124	0.33	28	14	New crack in the region of constant moment at midspan.a secondary slip crack occurred.
	42	39.65	395.4	920	124	0.4	28	16	Steel yielded.
	48	53.43	613.6	1250	111	1.0	30	29	
	52	93.42	853	1563	108	1.5	31	44	Cracks stabilized.
	55	137.72	944.4	1803	108	>1.5	31	71	Sharp cracking sound was heard.
	56	157.28	753	1881	108	>>>	31	78	Flexural shear failure with longitudinal crack directed from the left point load to inside the region of constant moment.
	36	178.36	354.6	1499	108	>>>	31	77	Load dropped down after crushing
releasing the applied load	0	156.43	267.2	644	108	>>>	31	49	
releasing the external force	0	142.92	201	653	108	>>>	31	31	Upward load was applied from bottom to release the external cable

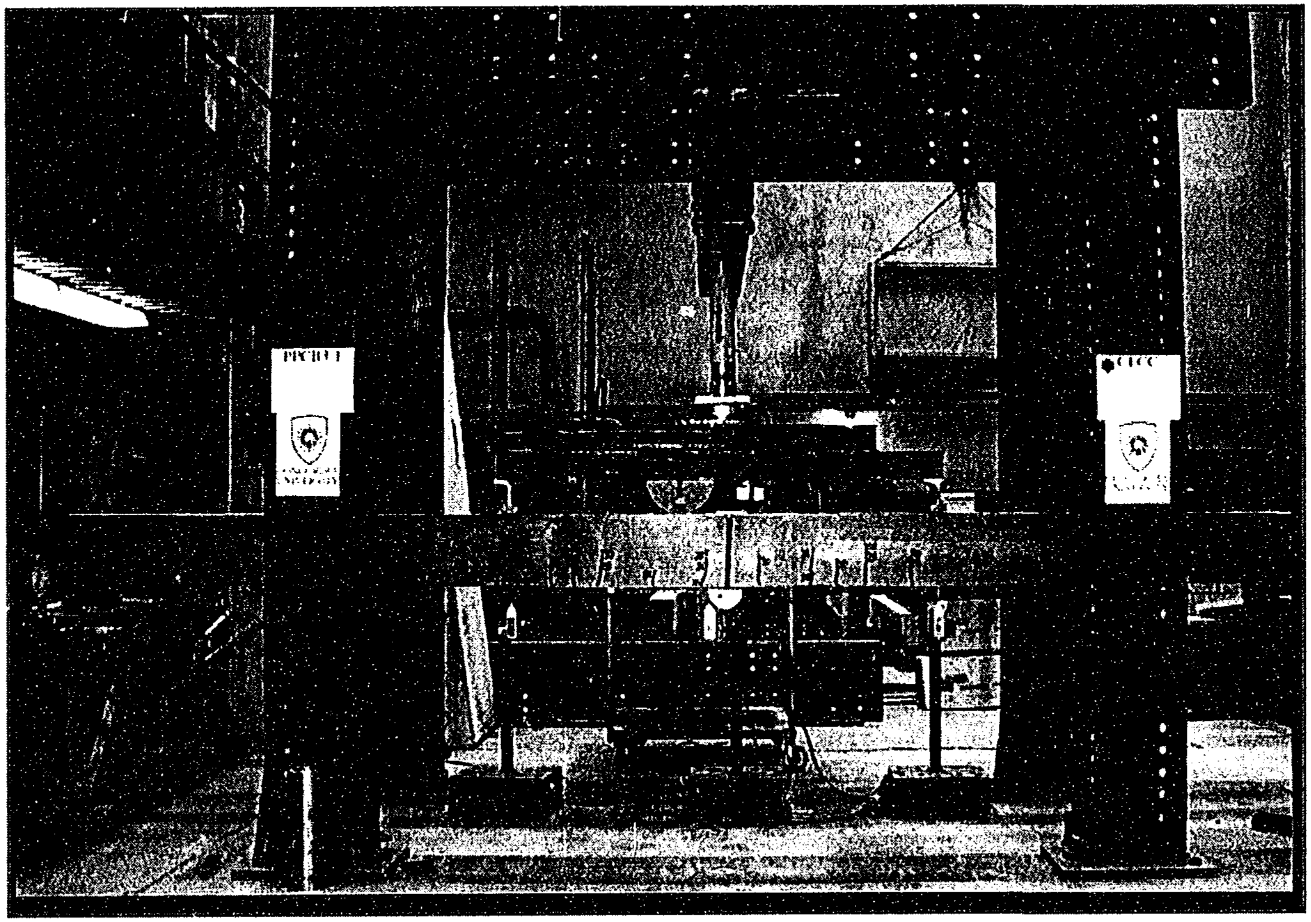


Figure A.25 Beam PPCB3-1 after Application of Initial Damaging Load

Figure A.26 Beam PPCB3-1 at Failure

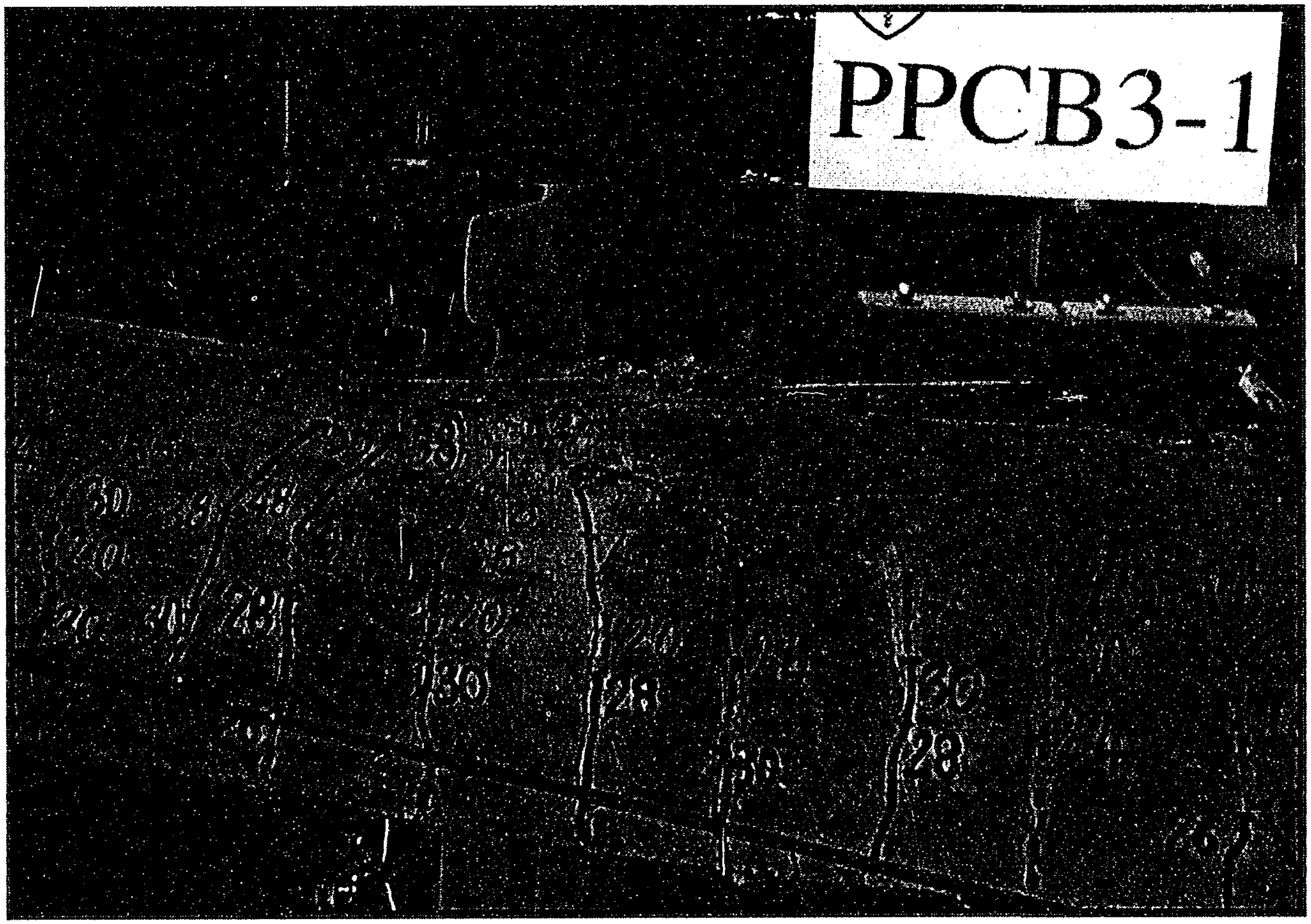


Figure A.27 Close-up of Failure of Beam PPCB3-1

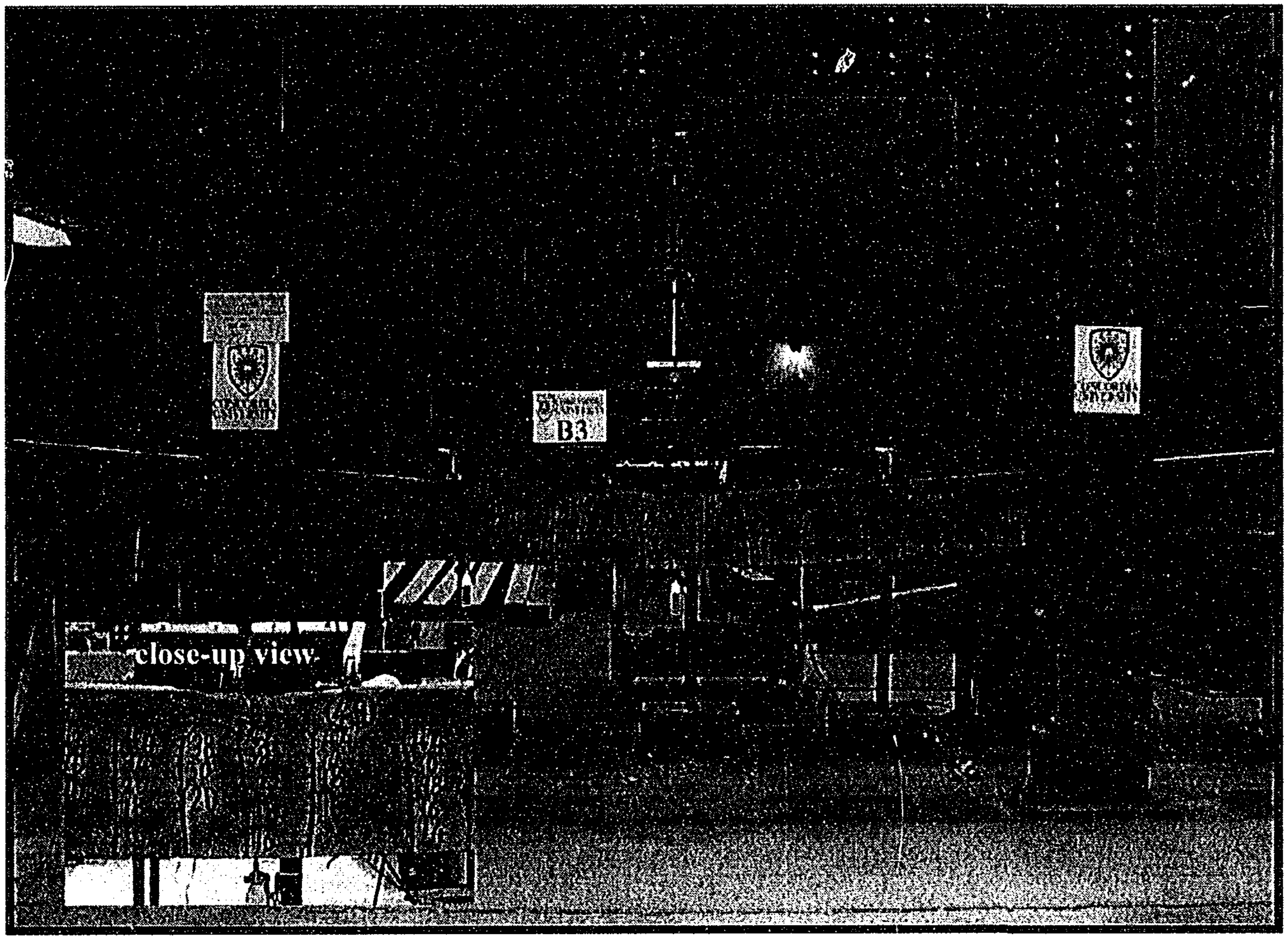


Figure A.28 Control Beam PPC3-1c at Failure

Table A.9a: Behaviour of Beam PPCB3-2 during Testing

L = 5.00 m S = 5.25 m $S/d_p = 125$ b = 150 mm h = 280 mm $d_s = 250$ mm $d_{ps} = 210$ mm $d_{pe} = 324$ mm	Ordinary Reinforcement: A_s = 2#10M (200 mm^2) $f_y = 400$ MPa $E_s = 200000$ MPa $A_s' =$ 2-6mm plain bars (56.62mm^2) $f_y' = 275$ MPa	Internal Prestressing: A_{ps} = 2-3/8 in strands (110 mm^2) $f_{pu} = 1860$ MPa $f_{py} = 0.9 f_{pu}$ (low relaxation) $f_{pe} = 0.6 f_{pu}$ $E_{ps} = 200000$ MPa	External Prestressing profile: one draped point at middle Type (1×7) CFCC 7.5mm $A_{pf} = 2(30.4) = 60.8mm^2$ $f_{puf} = 2040$ MPa $f_{pef} = 0.5 f_{pu}$ $E_{pf} = 135000$ MPa	$f_c' = 35$ MPa (Design) $f_c' = 42.5$ MPa (at time of testing) Moment at midspan M = PL/6

This beam was tested in a way different than the other beams:

a) The load was applied up to 46kN then removed before failure of the beam, (Table A.9b)
b) The load was applied until failure of the beam, (Table A.9c)

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
Initial damaging load	22	6.04	n/a	270	560	----	4	----	First crack.
	30	13.0		489	192	----	13	----	Cracks formed in the region of constant moment. Max. crack depth was 1/4 the beam depth.
	0	3.05		89	192	----	13	----	
sustained up to 74 days	1	1.08	n/a	147	192	----	13	----	under own weight only.

Table A.9b: Behaviour of Beam PPCB3-2 without Failure

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
reloading before external prestressing	0	4.13	n/a	236	192	----	13	----	Deflection, strain include residual values.
	30	23.5		866	105	----	27	----	New crack formed, and all cracks propagated to 1/2 beam depth.
external prestressing	29	23.5	n/a	866	105	----	27	0	
	28	22.25		892	105	----	27	32	Camber = 1.25mm
removing the applied load gradually	24	19.62	n/a	910	105	----	27	29	Deflection, strain include residual values.
	15	14.28		914	105	----	27	22	
	4	7.57		898	105	----	27	18	
increasing the applied load gradually	15	12.65	n/a	1088	105	----	27	24	
	24	18.57		945	105	----	27	30	
	40	30.8		1004	105	0.3	31	45	
	46	37.42		1159	107	----	32	53	Beam did not fail.
releasing the applied load	0	32.51	n/a	1268	107	----	32	17	
releasing the external prestressing	0	32.51	n/a	1301	107	----	32	0	

Table A.9c: Behaviour of Beam PPCB3-2 with Failure

Type of loading	Total applied load P (kN)	Deflection at midspan (mm)	Stress in reinforc. steel (MPa)	Top concrete strain (micro)	Average crack spacing (mm)	Max. crack width (mm)	Total number of cracks	Force in external CFCC cable (kN)	Remarks
reloading before external prestressing	0	4.13	n/a	256	105	----	27	----	After 17 days
	10	10.27		434	105	----	27	----	Deflection, strain include residual values
	20	18.96		645	105	----	27	----	
	30	27.62		855	105	0.25	27	----	
external prestressing	30	27.62	n/a	857	105	0.25	27	0	
	30	26.51		892	105	0.15	27	33	Camber = 1.11mm
reloading after strengthening	36	32.15	n/a	952	105	0.3	27	39	
	62	73.95		1720	104	0.8	37	99	
	72	97.84		2069	104	1.0	37	137	
	79	124.93		2275	104		37	180	Flexural failure by crushing under the left point load, and a longitudinal crack in the compression zone at the top directed inside the constant moment region
releasing the applied load and the external prestressing: due to a technical problem the deflection could not be measured and the force in the external cables was released by applying an upward load from the bottom									

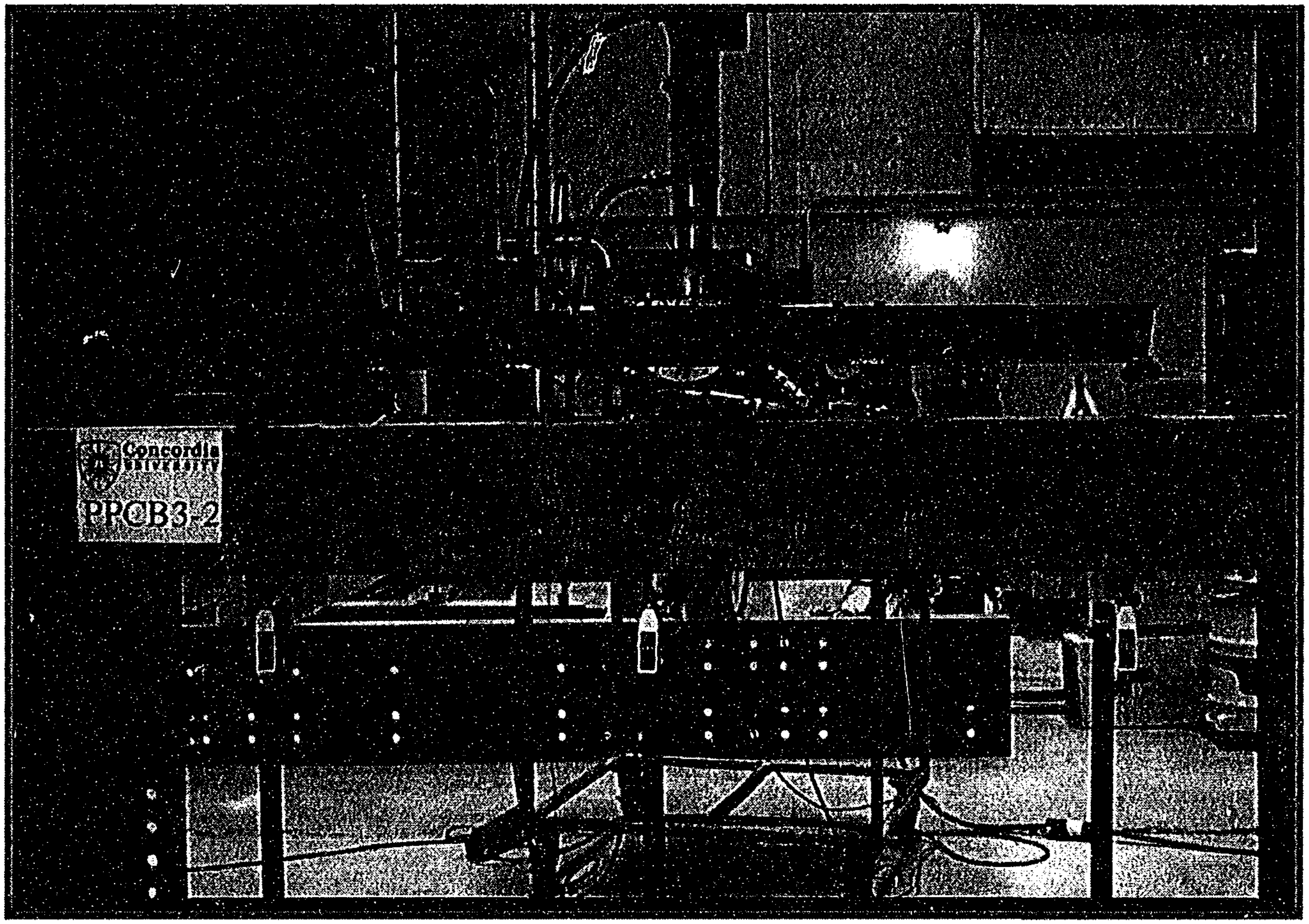


Figure A.29 Beam PPCB3-2 after Application of Initial Damaging Load

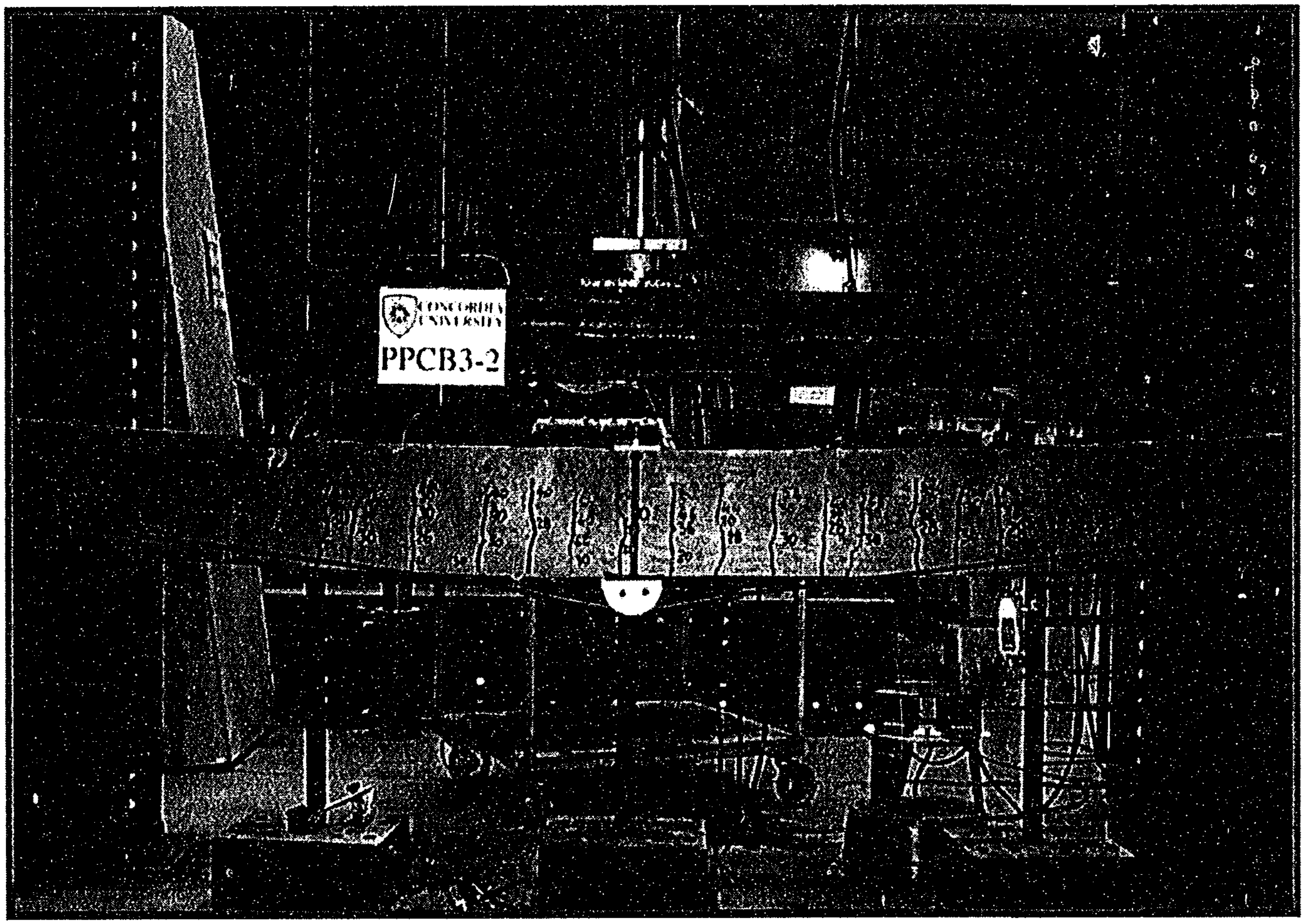


Figure A.30 Beam PPCB3-2 at Failure

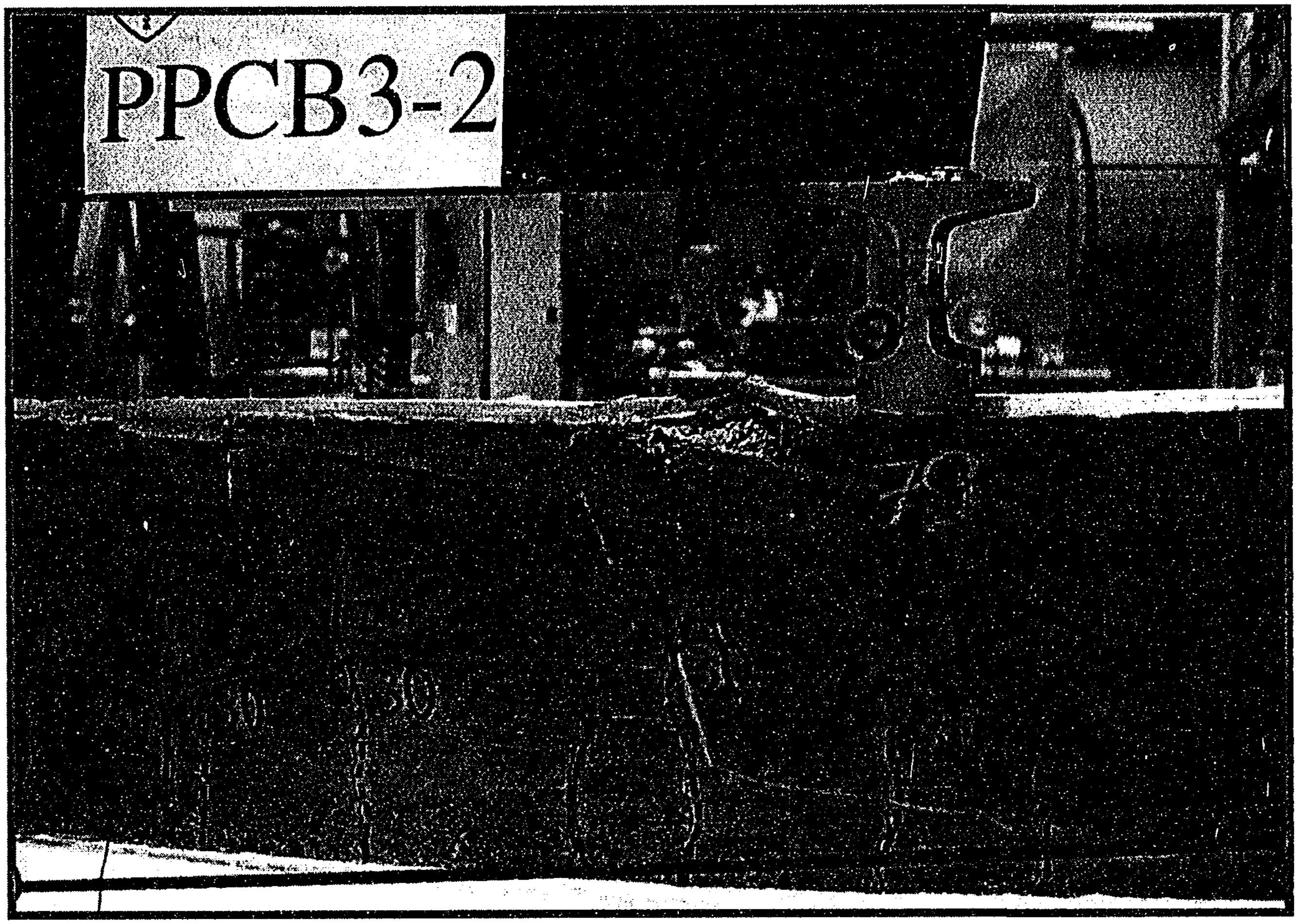


Figure A.31 Close-up of Failure of Beam PPCB3-2