## Abstract

The experimental evaluation of the CFRP stirrup strengths, bond strength of CFRP bars, transfer lengths of CFRP tendons, and shear and flexural responses of CFRP reinforced and prestressed concrete box-beams is presented. The shear and flexural design approaches are recommended. In addition, empirical stirrup strength design equations are proposed. This report consists of 6 chapters as given below:

Chapter 1-Introduction
Chapter 2-Literature Review
Chapter 3-Experimental Program
Chapter 4-Results and Discussion
Chapter 5-Design Guidelines and Examples
Chapter 6-Conclusions and Recommendations

## Keywords

Bonded and Unbonded Tendons, Box-beams, Bond Strength, CFCC, CFTC, CFRP, Concrete, Pretensioning, Post-tensioning, Prestressing, Stirrups, Strength, Transfer length

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FRP COMPOSITE PRESTRESSING STRANDS

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Stirrups  
Transfer Length

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ABSTRACT

Recently, the demand for using Carbon Fiber Reinforced Polymers (CFRP) in construction industry had gained momentum, however, as a new material, their properties and behavior have not been fully explored. This report presents experimental and analytical investigation of shear and flexural responses of CFRP reinforced and prestressed concrete box-beams. In addition, CFRP stirrup strengths, bond strengths of CFRP bars and tendons, and transfer lengths of CFRP tendons were experimentally evaluated and presented. Empirical design equations for the evaluation of stirrup strengths were formulated. This project was executed in various stages such as evaluation of stirrup strengths and bond characteristics of CFRP rods and strands, testing of box-beams prestressed using seven bonded pretensioning and six unbonded post-tensioning tendons for transfer length measurements, and for predicting the shear and flexural responses.

The experimental program consisted of testing one hundred eight specimens of rectangular shaped stirrups with two different end-anchorage configurations. The first end-anchorage configuration, type A, consisted of standard hook end, while the second configuration, type B, consisted of a continuous end. Stirrups with 7.5, 9.5, and 10.5 mm (0.3, 0.374, and 0.41 in.) diameters were used. Tested stirrups were made of carbon fiber twisted cable (CFTC), carbon fiber composite cable (CFCC), and CFRP bars provided by Diversified Composites, Inc. (DCI), Tokyo Rope Mfg. Co. Ltd., and Marshall Industries Composites, Inc. (MIC), respectively. The configuration of CFTC stirrup was similar to that of CFCC stirrups. In addition, a total of nine box-beams were constructed. Six box-beams were instrumented and tested for shear response, while the remaining three box-beams were instrumented and tested for flexure response. The six box-beams tested for shear and the two of the three box-beams tested for flexure were prestressed using seven bonded pretensioning and six unbonded post-tensioning tendons. The third box-beam tested for flexure was prestressed using only seven bonded pretensioning tendons, while the unbonded post-tensioning tendons were installed without prestressing, with anchor heads attached to both ends of the beam. Transfer lengths of pretensioning tendons were
measured for all nine box-beams. Independent four-point loading beams were used to examine shear and flexural responses. The analytical investigation consisted of development of nonlinear computer program to predict deflection, strain, and force in the unbonded post-tensioning tendons at various load levels. Comparing the analytical and experimental flexural responses of the box-beams validated the accuracy of computer program. A parametric study was conducted to examine the effect of the level of pretensioning and post-tensioning forces on the overall flexural response and ultimate load carrying capacity of the box-beams. In addition, a shear design approach is also presented to evaluate the shear strength of prestressed concrete box-beams.

To examine the bond characteristics of CFRP bars/tendons, twenty-five T-beams were tested. Three types of CFRP bars were used: MIC, DCI and Leadline, manufactured by Marshall Industries Composites Inc., Diversified Composites Inc., and Mitsubishi Chemical Corporation, respectively. The diameter of MIC and DCI bars was 0.374 in. (9.5 mm) while that of Leadline was 0.394 in. (10 mm). The measured bond strengths of DCI, MIC, and Leadline were 7.9, 6.9, and 6.3 MPa (1.15, 1.0, and 0.91 ksi), respectively, while that of 9.5 mm (0.374 in.) diameter conventional steel bar was 14.5 MPa (2.1 ksi).

From the detailed experimental investigation, it is concluded that the uni-axial stirrup strengths are lower than the strengths of strands in the direction parallel to the fibers. The embedded length, anchorage type, and tail length of stirrup of a specific bend radius and diameter affect the stirrup strength. The effective stirrup stress model in conjunction with uni-axial strength model estimate the shear strength of box-beams with reasonable accuracy. The difference in the theoretical and experimental values of the shear strengths of box-beams is less than 2 percent. It was observed that all the box-beams designed to fail in shear, failed due to widening of shear cracks followed by rupture of the pre-tensioning tendons due to dowel action. However, the CFRP stirrups did not rupture at the beam failure. Moreover, the combination of bonded and unbonded prestressing significantly increased the ultimate moment capacity of box-beams. For the same tendons, the beam with no initial post-tensioning prestress exhibited 20% lower
load carrying capacity than the beam prestressed with both the bonded pretensioning and unbonded post-tensioning tendons.

Flexural failure of all the box-beams was due to rupture of the bonded prestressing tendons followed by the crushing of concrete. Unbonded post-tensioning tendons remained intact without rupture even after the ultimate failure of the beams. However, the prestressing with unbonded post-tensioning tendons was observed to be ineffective with regard to improving the ductility of the box-beams due to under-reinforced box-beam sections. The level of initial pretensioning and post-tensioning forces significantly affects the flexural response of the beam and the failure mode.
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1.1 Statement of Problem

Corrosion of reinforcing steel bars in concrete has been a major problem of concern for the construction of civil engineering infrastructures such as bridges, and bridge-girders/beams especially in the cold countries. The de-icing salts and aggressive environmental conditions have led to severe deterioration of these infrastructures over the age. However, due to advent of advanced fibrous composite materials, such as carbon fiber reinforced polymers (CFRP), aramid fiber reinforced polymers (AFRP), and glass fiber reinforced polymers (GFRP), the problem of corrosion has been alleviated. Moreover, these fiber reinforced polymers have other notable characteristics such as high-strength and stiffness to weight ratios, lightweight, insensitive to magnetic effects, easy to handle, and fabricate laminates and/or lamina of desired strength and stiffness. In spite of several advantages offered by FRP materials, the variations and inconsistencies in their mechanical characteristics and their brittleness are still the major problem for establishing a unified design guidelines and codes.

Currently, worldwide research is going on to find suitable FRP materials such as FRP reinforcing bars, prestressing tendons/strands, plates, and sheets for internally and externally reinforced and prestressed concrete structures and to derive best possible structural efficiency in terms of structural strength and life with little maintenance. Although all the FRP materials have added advantages over the conventional materials, CFRP are being used widely for civil engineering structures due to its superior strength, stiffness, and durability qualities in comparison to other FRPs. The main characteristics of CFRP are given in the following section.
1.2 CFRP Characteristics

The strength to weight ratio of CFRP varies from 5 to 10 times that of steel and 50 times that of concrete. Similarly, the stiffness to weight ratio of CFRP is about 3 times that of steel and 25 times that of concrete. Moreover, the high resistant to corrosion, fatigue, and the best durability characteristics allow its use under adverse environmental condition with little maintenance. Similarly, the non-magnetic characteristics of CFRP allow its usage in some special structures like hospital and testing laboratories.

1.3 Project Objectives

To evaluate the mechanical characteristics of CFRP strands/tendons and stirrups, transfer length, bond characteristics of CFRP bars, and overall shear and flexural responses of the CFRP prestressed concrete box-beams, an experimental investigation was carried out with the following objectives.

- To fabricate the CFRP strand and stirrup specimens and to evaluate the mechanical characteristics of CFRP strands/tendons.
- To examine the effect of bend, tail length, embedment length, and anchorage configuration on the uni-axial strength of stirrups and to develop empirical design equations for predicting the stirrup strength.
- To measure the transfer length of CFRP tendons furnished by Diversified Composites, Inc. (DCI) and Mitsubishi Chemical Corporation (MCC) and compare it with corresponding transfer length obtained from the theoretical equation available in literature.
- To examine the bond characteristics of CFRP bars/tendons using shallow depth T-beam test specimens.
- To study the shear response, shear load carrying capacity, shear failure modes, cracking width, stirrup strain, and shear cracking loads of box-beams prestressed using bonded pretensioning and unbonded post-tensioning tendons with various arrangement and types of stirrups.
• To study the flexural response and load carrying capacity of box-beams reinforced with CFCC stirrups and prestressed independently using bonded pretensioning and unbonded post-tensioning DCI and Leadline tendons.
• To examine the energy ratios of box-beams tested for shear and flexural responses as measure of ductility of the beams.
• To develop shear and flexural design approach for box-beams prestressed using bonded and unbonded CFRP tendons.
• To develop nonlinear computer program to predict the deflection, strain, forces in unbonded post-tensioning tendons, ultimate flexural load carrying capacity of box-beams, and to conduct a parametric study to examine the effect of the level of pretensioning and post-tensioning forces on the flexural response of the beams.

1.4 Scope of the Work

This investigation consisted of experimental and analytical studies. The extensive experimental investigation was conducted to evaluate the mechanical characteristics of CFRP strands/tendons, behavior of CFRP stirrups under uni-axial and beam loading conditions, and shear and flexural responses of box-beams reinforced and prestressed using CFRP bars and tendons. Based on the experimental results empirical design equations were developed to determine uni-axial strength of CFRP stirrups, which was further used to determine the shear strength contribution of CFRP stirrups in CFRP prestressed concrete box-beams. Moreover, shear and flexural design approaches are also presented through design examples. The project report is further divided into following chapters:

Chapter 2: This chapter consists of a detailed literature review related to FRPs and FRP reinforced and prestressed concrete structures.
Chapter 3: This chapter consists of details of experimental program such as fabrication of test specimens, instrumentation, test setup, and procedure.
Chapter 4: This chapter consists of presentation and discussion of the experimental results.
Chapter 5: Flexural and shear design approaches are presented through design examples. In order to verify the proposed design approach, it was deemed necessary to use the dimensions, prestressing configuration, material properties, and reinforcing details of the tested box-beams. This facilitated a direct comparison between the theoretical and experimental results. Moreover, a parametric study is also presented based on a home-developed special purpose non-linear computer program.

Chapter 6: This chapter consists of the summary of concluding remarks and recommendations. Moreover, an appendix is provided at the end of the report, wherein appropriate notations used in the report are listed and equations for calculating the bond strength (from the measured bond failure load) of CFRP bars/tendons are presented.
2.1 General

Early researchers have reported on the development of innovative structural systems using advanced fibrous composite materials such as carbon fiber reinforced polymers (CFRP). Most have dealt with the use of CFRP as a longitudinal reinforcement in various structures and bridges (Rizkalla and Labossiere, 1999, ACI Committee 440, 1996, Dolan, 1999, and Grace et al., 2002). However, the development and the use of CFRP as shear reinforcement in the form of stirrups has not been studied enough to develop and formulate design guidelines. Moreover, the shear and flexural responses of concrete structures reinforced with CFRP stirrups and bars and prestressed using CFRP bonded pretensioning and unbonded post-tensioning tendons have not been examined in detail. Most of the available literatures related to the strength and response of CFRP stirrups, and shear and flexural responses of prestressed concrete structures are presented in the following sections.

2.2 CFRP Stirrup Strengths

Based on their experimental study, Maruyama et al. (1989) stated that the ultimate strength as determined from equal diameter bars cannot be developed in stirrups because diagonal shear cracks induce tensile forces which are oriented at an angle with respect to the stirrup. In a similar study on the tensile strength of the bent portion of FRP rods, Maruyama et al. (1993) concluded that the bent portion is more vulnerable to failure; the tensile strength of bent portion decreases as the curvature of the bend increases. Maruyama et al. (1993) also concluded that parameters such as concrete strength, fiber type, and rod manufacturing method influence the tensile strength of the bent rod.

Mochizuki et al. (1989), Nagaska et al. (1989), and Currier (1993) investigated the effects of bends in stirrups on their strength and concluded that bends can lead to a significant reduction in stirrup strength. Based on his push-out test of nylon/carbon stirrups embedded in concrete blocks, Currier (1993), specifically mentioned that the
capacity of stirrups reduced by 75% due to bend effects and the primary cause of failure of stirrup was the stress concentration at the bend portion.

Nakamura and Higai (1995) formulated an equation for evaluating the bend capacity of the stirrups in terms of guaranteed tensile strength of bar in a direction parallel to the fibers, radius of bend, and diameter of bars. Ueda et al. (1995) rationally and experimentally evaluated the strength and failure modes of 6 mm diameter closed AFRP stirrups. The study of Ueda et al. (1995) focused on the effect of embedment length on the stirrup strength and two-dimensional non-linear finite element analysis to study the local stresses at the bend portion. They concluded that higher embedment length allows higher stress development and the mode of failure of stirrups depended on the embedment length, i.e., the stirrups with 110 mm embedment length ruptured at straight portion, while the stirrups with 60 and 10 mm embedment lengths ruptured at bend portion. Ishihara et al. (1997) also conducted experimental and analytical studies using similar test setup as used by Ueda et al. (1995) to evaluate the effect of bend radius and Young’s modulus on the tensile strength of AFRP and CFRP rods. The study of Ishihara et al. (1995) resulted into an analytical expression to predict the strength of the bent portion of the rods.

Recently, Morphy et al. (1997) examined the performance of CFRP stirrups for concrete structures and studied the response of 5 and 7.5 mm diameter carbon fiber composite cables (CFCC) stirrups (provided by Tokyo Rope Mfg. Co. Ltd., 1993) and Leadline stirrups (provided by Mitsubishi Chemical Corporation, 1994) of a $5 \times 10$ mm rectangular cross-section. Test results indicated that the strength of the stirrups could be as low as 40 percent of the ultimate tensile strength of the material parallel to the fibers as determined from equal diameter tensile bars. In addition, based on beam tests, Morphy et al. (1997) recommended limiting design strength of CFRP stirrups to 50% of the tensile strength parallel to the fibers.

Some researchers such as Maruyama et al. (1989) and Nakamura and Higai (1995) investigated the effect of kinking of stirrups on the stirrup strength. The kinking of stirrups is also major issue of concern, because the strength of bar in a direction other
than fiber direction is much less than that parallel to fiber direction. The effect of kinking refers to a condition wherein an inclined crack intersects the stirrups and stirrup is subjected to diagonal tensile stress. According to Maruyama et al. (1989), the diagonal tensile strength of bars was reduced by 70%, 55%, and 35% in the cases of CFRP, AFRP, and GFRP stirrups, respectively.

Kanematsu et al. (1993) and Ueda et al. (1995) conducted analytical and experimental studies to evaluate the AFRP stirrup strength subjected to tensile and shear forces. They studied the effect of crack width on the stirrups tensile strength and concluded that the stirrups strengths are reduced by the crack width and the shear displacement. They also concluded that the debonding length around the bar at the crack location, bond stress-slip relationship, shear modulus and strength of bars have significant effect on the stirrup strengths.

The research investigation by Fam et al. (1995) and report (ACI 440.1R, 2001) demonstrate that the effective strain/stress in stirrups could be used for evaluating the shear strength contribution of stirrups used in reinforced and prestressed concrete beams. However, no specific effective CFRP stirrup strength model is available in literature, which can be used in conjunction with uni-axial stirrup strengths for shear strength design of reinforced and prestressed concrete beams.

2.3 Shear Behavior of Reinforced Concrete (RC) Beams

Despite many years of experimental research and established design guidelines, the shear transfer mechanism of concrete is still not well understood. The shear failure in reinforced concrete beams is sudden and disastrous, which requires that flexural failure should precede shear failure of any concrete structure. The shear response and shear strength of concrete beams primarily depend on shear span to depth ratio, compressive strength of concrete, longitudinal reinforcement ratio, shear reinforcement ratio, type of loading, size of beam cross-section, and the magnitude of effective prestress, if any (Shehata, 1999). The other references on the mechanism of shear transfer and shear
strength contribution of reinforced and/or prestressed concrete beams are presented below.

In an experimental investigation conducted on a test beam at Cornell University, Nilsen and Winter (1991) showed that beams without shear reinforcement failed immediately upon formation of a diagonal crack in the critical shear zone adjacent to the support. It was recommended that shear reinforcement is prerequisite in the beams to avoid premature shear failure. It was also emphasized that shear forces acting on a beam is resisted by the un-cracked portion of the concrete, interlock between aggregates, and the longitudinal reinforcements resisting dowel action, and the shear reinforcements.

It was also concluded (Nilsen and Winter, 1991) that the shear reinforcement contributes to the shear resistance only after diagonal shear cracks appear. The shear reinforcements add to the shear resistance after the cracking of concrete in five different ways such as (i) bars that intersect a particular crack resist part of the force, (ii) presence of shear reinforcement prevents the advancement of diagonal cracks into the compression zone, thereby, leaving more uncrazed concrete section for resisting shear force, (iii) stirrups ties the longitudinal reinforcement in bulk, enhancing the confinement of the concrete, (iv) preventing the widening of the crack, thereby improving the interlock between the aggregates, and (v) support the longitudinal reinforcement, hence resists the dowel action.

Park and Paulay (1993) hypothesized the analogy between the shear behavior of a parallel chord truss and web-reinforced concrete beam. Their suggested truss model had the stirrups acting as tension members and concrete struts running parallel to the diagonal cracks. The compression zone concrete acts as the top chord and flexural tension reinforcement acts as bottom chord of the hypothetical pin-jointed truss.

2.4 Shear Behavior of FRP Reinforced Concrete Beams

The shear behavior of concrete beams reinforced with FRP is different from the beams reinforced with steel rebars due to facts that unlike steel, FRP materials are linearly elastic until failure and have low stiffness and strength in direction normal to the
fiber direction. Hence, dowel action plays a very important role in limiting the shear strength of FRP reinforced beams. Zhao et al. (1995) studied the shear behavior of concrete beams reinforced with FRP rods as longitudinal reinforcement and closed loop GFRP and CFRP stirrups. It was observed that the most of the specimens failed due to shear-compression failure and the higher stiffness of stirrups resulted in the higher shear capacity of the beams with smaller stirrup strain at failure. Zhao et al. (1995) also studied the shear strength contribution of FRP stirrups in terms of stirrup strains, crack width, and shear deformation.

Maruyama and Zhao (1996) studied the effect of the size of the beams on the shear response of the FRP reinforced concrete beams. The experimental program consisted of testing five concrete beams of different sizes but of the same width to the depth ratio and reinforcements. Two of the five beams were without shear reinforcements, while remaining three beams were provided with CFRP stirrups. From the measured strains in the concrete, and longitudinal and shear reinforcements, and measured crack width, it was concluded that the size effect law (JSCE, 1997) is valid for the CFRP reinforced beams.

The study of Vijay et al. (1996) on the shear capacity of beams reinforced with open loop CFRP stirrups concluded that the shear failure in concrete beams are either governed by stirrup rupture or bond failure and the ACI Eq. 11-17 (ACI 318, 2002) is valid for computing the shear strength contribution of FRP stirrups. It was also concluded that the effective embedment lengths and developed bond stresses are important for evaluation of permissible design stresses.

Alsayed et al. (1997) tested 21 full-size beams reinforced with different combinations of steel and GFRP for longitudinal and transverse reinforcements. They suggested different analytical models for nominal shear strength of beams for different combination of transverse and longitudinal reinforcements fabricated from GFRP and steel.
Based on the study of shear behavior of concrete beams reinforced with CFRP and AFRP bars and AFRP preformed spiral stirrups, Yonekura et al. (1993) developed analytical models for predicting the ultimate shear strength of beams with factor of safety greater than 1. They concluded that the beams in the shear tests failed in either shear compression or due to stirrup rupture. Nagaska et al. (1993) studied the shear performance of concrete beams reinforced with AFRP and CFRP preformed stirrups. Based on tests conducted on 35 half-scaled beams, they concluded that the shear strength of beams depends on two predominant failure modes such as rupture of the stirrups at the bent and crushing of concrete. The failure mode distinguished by shear reinforcement ratio, which is function of stirrup reinforcement ratio, bend capacity, and compressive strength of concrete. It was also concluded that the shear capacity of the beam is affected by the axial rigidity of stirrups.

Based on past experimental researches, Sato et al. (1994,1995) studied the ultimate shear response of FRP reinforced concrete beams. They developed a unified shear model for beams with and without shear reinforcements using modified shear resisting mechanism to predict diagonal tension strengths. Choi et al. (1997) predicted shear-resisting mechanism of FRP reinforced concrete beams by lattice model and checked the validity of truss analogy. The lattice model consisted of molding concrete into a flexural compression member, flexural tension member, diagonal tension member, and an arch member and molding reinforcement into horizontal and vertical members.

2.5 Shear Behavior of FRP Prestressed Concrete Beams

Yonekura et al. (1993) studied the shear behavior of prestressed concrete beams prestressed using CFRP and AFRP tendons and compared the results with that of similar beams reinforced with steel. It was concluded that the shear strength of beams prestressed and reinforced with FRP for flexure and shear is lower than that using steel reinforcement, provided the shear strength contributed by the web reinforcement is same in both cases. Yonekura et al. (1993) also concluded that increasing the prestressing force is an effective way to increase the shear strength of FRP reinforced beams. Moreover, their study revealed that the small increase in the shear reinforcement changes
the failure mode from shear failure to flexural failure, except in the case of FRP reinforced beams.

The shear capacity of prestressed concrete beam was also studied by Tottori and Wakui (1993). The effective prestressing force was computed using decompression moment and from the test results, it was concluded that the type of prestressing tendons does not affect the shear capacity of beams. Fam et al. (1995) also studied the response of CFRP prestressed concrete bridge-beams reinforced with CFRP bars and stirrups for flexure and shear, respectively. They observed shear failure of beams with spalling of concrete cover at the bent portion of stirrups. They concluded that the maximum developed shear strain in the Leadline stirrups was 0.85%, while the stirrup strength was 45% of the uni-axial tensile strength.

Based on the study of shear response of CFRP prestressed concrete beams, Naaman and Park (1997) concluded that the dowel action at the critical shear cracked plane caused the premature failure of the beams due to rupture of CFRP tendons. Unlike steel reinforced beams, the dowel action is effective in CFRP reinforced structures due to low transverse strength and brittleness of the CFRP tendons/bars. It was also concluded that the beams prestressed with steel tendons have higher shear capacity than that with FRP for the same reinforcing index. Moreover, it was observed that adding fibers to concrete matrix increases the shear capacity of the beams by delaying the shear tendon rupture, and the shear crack width and displacement are lower than that with steel.

Dowden and Dolan (1997) compared the available experimental results (Yonekura et al., 1993) with that predicted by established codes, ACI 318 (2002) and AASHTO (1998) and concluded that the flexure-shear and web-shear criteria (ACI 318, 2002) over predicted the experimental results. The mean ratio of experimental to theoretical shear strengths was 0.77. Similarly, AASHTO code (1998) predicted higher values of the shear strength in comparison to that of experimental. The experimental to theoretical strengths ranged from 0.337 to 0.510.
2.6 Bond Behavior of FRP Bars and Tendons

Since the transfer of stresses between concrete and reinforcing bars under bending and the crack control depend on the quality of bond, the surface of FRP bars has to be specially prepared to develop sufficient bond stresses and to avoid slippage of reinforcing bars. The adequate bond stresses at the interface between the concrete and FRP result in utilizing the full strength of structures. Some of the early studies related to bond behavior of bars obtained through beam and pullout tests are summarized in the following sections.

2.6.1 Beam Tests

Makitani et al. (1993) experimentally investigated the bond behavior of CFRP, AFRP, GFRP, and Vinlylyn using beam tests. It was observed that for a bond length 40 times the diameter of bar, sufficient bond stresses developed to attain full tensile strength of bars. They also noted that the bond strength tends to increase with the increase in the bond length. Similarly, molding the surface of the bars in a spiral shaped ribs or covering the bars with sand improves the bond capacity of the bars. Sixteen concrete beams reinforced with GFRP bars were tested by Benmokrane et al. (1994) in flexure to evaluate the bond strength of GFRP bar splices. It was observed that the full tensile strength of GFRP bars developed for embedment length equal to 1.6 times the development length (ACI 318, 2002).

Takagi et al. (1997) studied the bond of continuous braided aramid fiber rods and carbon fiber rods using beam specimens subjected to static and fatigue loads as per the procedures of ACI Committee 208 and RILEM CEB-FIP Committee. They studied the effect of prestressing also and stated that the bond strength is significantly affected by the method of loading, presence of prestressing force, and the variation of embedment length. It was also noted that for the AFRP bars, load corresponding to the bar slippage increased with application of prestressing force, however, this effect was not observed in the case of CFRP bars.
2.6.2 Pullout Tests

Ehasni et al. (1993) investigated the effect of compressive strength of concrete, diameter of bar, clear concrete cover, radius of bend, and hook extension on the failure mode and bond strength of GFRP bars using pull-out tests on 20 specimens of ordinary strength concrete. It was observed that the bond strength of GFRP bars is lower than that of steel bars. Moreover, hooked bars with larger bend radius provided larger stiffness, however, the increase in the hook length beyond 12 times the diameter of bar did not increase the hook strength. From the pullout tests of Hattori et al. (1995) to examine the creep behavior of GFRP and CFRP bars embedded in concrete under sustained loading, it was concluded that the mechanical properties of resins affect the bond-slip behavior of bars. It was noted that the AFRP bars exhibit larger free end slip compared to steel bars. However, the bond creep effect in beams reinforced with carbon fiber twisted cables (CFTC) and AFRP bars were not significant.

Jerret and Ahmad (1995) conducted pull-out bond tests on three smooth and three deformed 8 mm diameter CFRP bars embedded into concrete slab. The average bond strength of smooth CFRP bars was observed to be very low compared to their ultimate strength, while the deformed bars exhibited bond strengths of 2 to 5 times that of smooth bars. It was noted that smooth bars reached the maximum pullout load without free end slippage, however, the deformed bars showed a significant increase in the pullout load after initial free end displacement. Based on the direct pullout tests conducted on both the smooth and machined GFRP and CFRP bars embedded into concrete cubes, Nanni et al. (1995) concluded that the strength and mechanical action on the surface of FRP rods rather than the adhesion and friction between concrete and bars control FRP bond capacity. All the machined bars failed due to the shear-off of the rings due to sliding. It was noted that the compressive strength of concrete had little effect on the bond strength of the FRP bars and no concrete failure was detected.

Achillides et al. (1997) examined the effect of FRP type, shape, FRP bar diameter, and concrete compressive strength on the bond behavior using pullout tests on CFRP and GFRP bars embedded into concrete. Authors concluded that the nonlinear
distribution of bond stresses results in lower average bond stresses with increase in the embedment length. They also concluded that the bond strength of FRP bars is independent of the concrete compressive strength, conversely, bond strength is totally dependent on the shear strength of the resin of the bar due to peeling of surface layers of bars. They also observed that larger diameter bars resulted into lower bond strength and the shape of FRP bars has little effect on the their bond strength.

The thermal and mechanical effects on bond between GFRP bar and concrete were studied by Shield et al. (1997) using 30 inverted half-beam specimens. The effects of the bar diameter and material properties were investigated. Six half-beam specimens were subjected to mechanical fatigue, twelve were subjected to thermal fatigue, and the remaining twelve were tested under static loading. They observed that the mechanical fatigue did not affect the bond strength of GFRP bars, while it reduced the bond strength of steel bars by about 13%. However, thermal test cycles reduced the bond strength of GFRP bars more than that of steel, i.e., reductions in bond strength of GFRP and steel bars due to thermal fatigue were 12% and 3%, respectively.

Wang et al. (1997) studied the bond characteristics of GFRP, CFRP, and AFRP, and steel bars using pullout tests and examined the effect of fiber type, surface configuration, and modulus of elasticity on the FRP bond strength. They stated that the GFRP bars exhibited the highest bond strength, while CFRP and AFRP bars had no appreciable difference in their bond strengths. They also noted that the shapes of the bar surfaces, spiral, deformed, and braid, did not show the difference in the bond capacity, however, smooth surfaces yielded a very small bond strength value. It was also noted that for the same surface configuration, bars with higher elastic modulus had higher bond strength.

In addition to the above experimental studies, Cosenza et al. (1995) analytically modeled the bond behavior between FRP bars and concrete and have discussed the other existing models. Based on their analytical study, they concluded that the bond between the FRP and concrete depends on the surface configuration of bar and the manufacturing
process. It was also stated that the grain-covered bars provide the best results in terms of bond strength.

2.7 Flexural Behavior of CFRP Prestressed and Reinforced Concrete Beams

The flexural response of prestressed concrete structures depends on the type of prestressing such as pretensioning and post-tensioning, level of prestressing forces, type of prestressing tendons, and reinforcement ratio. Moreover, transfer length, which is an important parameter for structural design and construction of pretensioned prestressed concrete structures. The transfer length determines the location along the beam where the prestressing force is in full effect and is defined as the length from the free end of the beam to the point where full prestressing force is transferred to the concrete from the released pretensioning tendons. The early research investigations related to the transfer length, and flexural response of prestressed concrete structures using bonded pretensioning, and unbonded post-tensioning tendons/strands are discussed in the following sections:

2.7.1 Transfer Length

Nanni et al. (1992) experimentally examined the transfer length of braided AFRP tendons using twenty-five test beams. They concluded that the friction was the predominant bonding mechanism in AFRP tendons and these tendons show little slippage when compared to steel strands. Based on the measured transfer lengths for seven specimens pretensioned using 3/8 in. GFRP (S-2 glass fiber) strands and five specimens pretensioned using 0.5 in. steel strand, Issa et al. (1993) concluded that the transfer length is about 28 times the nominal diameter of pretensioning strand. They also concluded that the GFRP strands had better bond characteristics than steel due to better adhesion and interlock at transfer.

Abdelrahman (1995) experimentally measured the transfer length of 8 mm (5/16 in.) diameter CFRP Leadline tendons using prestressing levels of 50 and 70% of the guaranteed strength. The author concluded that the transfer length depends on the level of prestress in the Leadline tendons. Similarly, Domenico (1995) examined the transfer
lengths and bond characteristics of twenty pretensioned concrete beams prestressed by seven wire CFCC strands of diameters varying from 1/2 to 5/8 in. (12.5 to 16 mm). He used different concrete covers 2 to 3 in. (50.8 to 76.2 mm) and strengths 5.36 to 10 ksi (37 to 69 MPa), and different prestressing levels. It was concluded that the transfer lengths is proportional to the diameter of strand and the level of prestressing.

Based on tests on 29 specimens prestressed with 0.2 and 0.3 in. (5.1 and 7.6 mm) diameter arapree aramid fiber tendons with different surface finishes, Taerwe and Pallemans (1995) suggested that transfer length of arapree aramid fiber tendons may be taken equal to 16 times their nominal diameter. Furthermore, Ehsani et al. (1997) examined the transfer lengths of 0.39 in. (10 mm) diameter Arapree, 0.41 in. (10.5 mm) diameter FiBRA, and 0.29 in. (7.5 mm) diameter Technora tendons, where Arapree tendons were round with sand-impregnated surface; the Fibra tendons were braided; and technora tendons were deformed with a spiral indentation. The authors noted that the level of prestressing significantly affected the transfer length of Arapree tendons. The measured transfer lengths for Arapree, FiBRA, and Technora tendons were 50, 33, and 43 times their nominal diameter, respectively.

Based on the transfer length measurements for concrete beams prestressed using pretensioning CFRP Leadline tendons and CFCC strands, Mahmoud et al. (1999) recommended the following transfer length model (Eq. 2.1) for CFRP Leadline tendon and CFCC strands.

\[
L_t = \frac{f_{pi} d_b}{\alpha_t f_{ci}}^{0.67} \text{ mm} \tag{2.1}
\]

where

\(L_t\) is the transfer length (mm); \(f_{pi}\) is prestress at transfer (MPa); \(d_b\) is nominal diameter (mm) of tendon/strand; and \(\alpha_t\) is transfer length coefficients. Here, \(\alpha_t\) equals 1.9 for CFRP Leadline tendons and 4.8 for CFCC strands. It should be noted that the above transfer length model is based on 5/16 in. (8 mm) diameter CFRP Leadline tendons,
CFCC strands of three different diameters, i.e., 7/16, 1/2, and 5/8 in. (10.5, 12.5, and 16 mm) different concrete strengths at transfer 3.2 to 5.1 ksi (22 to 35.2 MPa), and prestressing levels of 58 to 80 percent of the guaranteed strength.

Grace (2000a) also investigated the transfer lengths for double-Tee (DT) concrete beams pretensioned using 8 and 10 mm diameter CFRP Leadline tendons and 12.5 mm diameter CFCC strands and modified the transfer length coefficients of Mahmoud et al. (1999) model as expressed in Eq. 2.1. The transfer lengths coefficients recommended by Grace (2000a) are 1.95 for Leadline tendons and 2.12 for CFCC strands. However, it should be noted that the modified transfer length coefficients is based on a single grade of concrete having strength of 7 ksi (48 MPa) at transfer.

2.7.2 Flexural Response of Beams Prestressed with Bonded Pretensioning Tendons

Naaman et al. (1993) examined the flexural behavior of concrete beams partially prestressed with CFCC strands and emphasized that careful observation is needed during and after prestressing of beams. They concluded that the non-prestressing steel reinforcing bars can help in providing residual strength and ductility based on the observation that the failure of prestressing tendons occurred after the yielding of steel reinforcing bars. It was also concluded that the conventional methods of equilibrium, strain compatibility, and material constitutive relations could be used to predict the fully or partially prestressed beams using CFCC strands.

The effects of type and quantity of pretensioning tendons, axial reinforcement, and the level of initial prestressing forces on the flexural response of concrete I-beams prestressed using AFRP and CFRP tendons were examined by Yonekura et al. (1993). It was observed that the deflection of concrete beams reinforced and prestressed using FRP tendons is greater than that of those using steel bars.

Abdelrahman et al. (1997) analytically and experimentally evaluated the deformability of rectangular and T-sections flexural members prestressed with CFRP tendons and compared the results with that of beams prestressed with steel strands. It was concluded that CFRP prestressed concrete structures should be designed to have
considerable deformation and crack formation before failure. It was also recommended that the nominal strength of prestressed concrete members should be at least twice the cracking moment and the net tensile strain in the reinforcement should not exceed 0.005 at the nominal strength of the member. Moreover, it was stated that the ratio of neutral axis depth to the effective depth of the members should not exceed 0.4 unless the nominal moment capacity of member exceeds 1.5 times the factored moment. The energy dissipation of CFRP prestressed rectangular section beams was in close agreement with that of steel prestressed beams, however, the CFRP prestressed T-beams dissipated less energy in comparison to that prestressed with steel strands.

The creep, shrinkage, and prestress transfer characteristics of eight large scale CFRP Leadline prestressed T-beams were examined by Soudki et al. (1997). It was observed that under the self-weight, CFRP Leadline prestressed beams exhibited larger creep than that prestressed with steel strands.

Zou et al. (1997) studied the overall load-deflection response, residual deformation, and ductility of two CFRP prestressed rectangular section beams. It was observed that the load deflection behavior of the beams was linear before cracking, and bilinear after cracking, but with reduced stiffness. It was also observed that the deflection corresponding to the ultimate failure load reduced with increase in the concrete strength.

2.7.3 Flexural Response of Beams Prestressed with Bonded and Unbonded Tendons

Kato and Hayashida (1993) studied the flexural characteristics of concrete beams prestressed using bonded and unbonded CFRP tendons. It was concluded that failure of bonded CFRP prestressed concrete beams was brittle, whereas the beams prestressed with unbonded CFRP tendons had roughly the same degree of ductility as that of beams reinforced with steel rebars. It was noted that the cyclic loading have no significant effect on the flexural response of CFRP prestressed beams (Grace and Sayed, 1997, Grace, 2000b, and Grace et al., 2002).

Based on the experimental investigation on the flexural response of concrete T-beams prestressed using external AFRP, CFRP, and steel cables, Mutsuyoshi and
Machida (1993) concluded that the load-deflection responses of FRP prestressed beams are similar to those prestressed with steel. It was noted that the ductility of the FRP reinforced beams could be improved with the provision of external prestressing. Also, Maissen and De Smet (1995) compared the behavior of concrete beams prestressed using CFRP bonded and unbonded tendons with that of the beam prestresses with bonded steel strands. They concluded that the flexural capacity of beams prestressed with unbonded tendons is greater than that of the beam prestressed with bonded tendons.

The study of Naaman and Jeong (1995) on the structural ductility of concrete beams prestressed with AFRP, CFRP, and steel strands concluded that the beams prestressed with the FRP tendons have considerably low ductility in comparison to those beams prestressed with steel strands. It was mentioned that the provision of non-prestressing rebars and external post-tensioning could increase the ductility of the FRP prestressed beams.

Tezuka et al. (1995) conducted an experimental study to predict the moment redistribution of continuous beams pre-stressed using FRP pretensioning tendons. They tested five two-span continuous beams of rectangular cross-section. It was observed that significant change in the moment redistribution occurs due to large diagonal shear cracks.

The flexural behavior of double –T (DT) girders prestressed using internal bonded pretensioning and external post-tensioning CFRP tendons/strands was examined by Grace and Sayed (1997) with and without repeated load effects. It was observed that the repeated load has no effect on the bridge system and structural components and the eccentric load was well distributed between webs without being effected by the cyclic loading. The authors mentioned that the bridge system was reasonably ductile with an energy ratio of 62% at the ultimate load. Recently, Grace and Sayed (2002a,b) also evaluated the full-scale DT-beam reinforced and prestressed using CFRP/CFCC tendons/strands. Additional references on the responses of prestressed concrete structures can be obtained elsewhere (Grace, 1999, Grace et al., 1999, Grace and Sayed, 1998a, b, Grace and Sayed 1996).
Taniguchi et al. (1997) examined the flexural response of concrete beams prestressed externally using CFRP and AFRP tendons under static and dynamic loadings. The longitudinal and transverse reinforcements were either steel or CFRP. It was observed that the CFRP and AFRP ropes used as external prestressing strands broke at stress levels equal to 70% and 90% of their ultimate tensile strength, respectively. The use of CFRP as transverse reinforcement increased the ductility of the system. It was also observed that prestressing cables did not fail even after 2 million cycles of repeated loads and they register a tension below 50% of their ultimate strength.

A new proposed construction approach (Grace et al., 2001) for multi-span CFRP prestressed continuous concrete bridges recommends that the external post-tensioning using draped tendons, continuity of deck slab, and transverse post-tensioning increased the ductility of the bridge system. The ductility index of the continuous bridge system was observed to increase by 48%, while the midspan deflection was reduced by 75% as compared to simply supported bridge constructed with similar components. Grace et al. (2001) also concluded that the progressive failure of CFRP tendons results in the loss of strength and stiffness after the ultimate load.

Most recently, Grace and Singh (2002) have proposed a unified analysis and design approach for design of a CFRP prestressed concrete bridge beams/girders prestressed using bonded pretensioning and unbonded post-tensioning tendons arranged in multiple vertically distributed layers along with non-prestressing CFRP bars. Design equations to determine the flexural capacity and to compute the stresses and strains in the concrete and tendons are provided. In addition, Grace and Singh (2002a) also studied the overall response of beams such as deflections, strains, cracking loads, and post-tensioning forces using a developed computer program incorporating parabolic stress-strain relation for concrete and linear stress-strain relation for CFRP tendons.

The unified design approach of Grace and Singh (2002) was validated by comparing the analytical and experimental results (Grace and Sayed, 2002a). The experimental results (Grace and Sayed, 2002a) was obtained from a full-scale DT test beams. Based on a detailed parametric study (Grace and Singh, 2002), it was observed
that the reinforcement ratio and the level of prestressing have significant effect on the moment carrying capacity and ultimate load deflection of the beam. It was concluded that the combination of bonded and unbonded prestressing levels (0.3 to 0.6) could significantly increase the moment capacity of the beam.

2.8 The City of Southfield Bridge Street Bridge

The Bridge Street Bridge (Grace et al., 2002) consists of two separate parallel and independent bridges (Structures A and B) over the Rouge River in the City of Southfield, Michigan, (see Figures 2.1, 2.2, and 2.3). Both bridges comprise three spans skewed at 15 degrees over a 62 m (204 ft) length and carry traffic near an industrial subdivision. Structure A was constructed first, and it consists of a new substructure and a new superstructure that incorporates five equally spaced conventional AASHTO Type III precast concrete I-girders in each of the three spans, with a continuous cast-in-place concrete deck slab. Structure B consists of four special precast, prestressed double-tee (DT) girders in each of the three spans configured as simply supported spans. Each DT girder is structurally reinforced using pretensioned carbon fiber reinforced polymer (CFRP) Leadline tendons and post-tensioned carbon fiber composite cable (CFCC) strands in both longitudinal and transverse directions. The non-prestressed reinforcement in the girders and deck structure is a CFRP material manufactured in bent configuration (CFCC), straight CFRP reinforcing bars (CFCC), CFRP NEFMAC grid reinforcement, and stainless steel reinforcing bars for stirrups. Hollowcore Incorporated (HI), Windsor, Ontario, Canada, fabricated all the girders for both structures, while Construction Technology Laboratories, Inc. (CTL), Skokie, Illinois, installed the instrumentation for long-term monitoring. Hubell, Roth & Clark performed the design and construction engineering, inspection, and material testing, and support groups.

Ultimately, this project will demonstrate that the use of CFRP material as structural reinforcement can dramatically increase the service life of highway bridges, thereby reducing construction related safety concerns and annual maintenance costs. To optimize bridge durability even further, a quality review of materials was conducted and decisions were made to require very high quality concrete and to allow metallic reinforcement made only of stainless steel. The Bridge Street Bridge is the winner of the
prestigious Precast/Prestressed Concrete Institute’s Harry H. Edwards Award for Industry Advancement.

2.9 CFRP Prestressed Bridges in North America

In addition to the above described Bridge Street Bridge, City of Southfield, Michigan, FRP has been used for various bridge girders, deck slabs, and barrier walls in several new bridges of Canada due to severe environmental conditions and use of salt for deicing roads (Rizkalla and Labossiere, 1999). These projects were completed through networking and collaboration between ISIS Canada and various provincial and municipal highway departments across the country. It should be noted that more than 40% of the bridges operating in Canada were built over 30 years ago and most are in urgent need of replacement or rehabilitation. The following section reviews the design and construction of three bridges in Manitoba, Quebec, and Alberta (Rizkalla and Labossiere, 1999).

Taylor Bridge- The Taylor bridge was opened in 1997 by the Province of Manitoba’s Department of Highways and Transportation. This bridge includes both composite material reinforcements and remote monitoring using a fiber optic structural sensing system. This bridge is located on Provincial Road 334 over the Assiniboine River in Headingley, Manitoba. The total length of the bridge is 165 m (540 ft), divided into five equal spans. Each span consists of eight I-shaped precast prestressed concrete girders. Four out of the 40 precast girders are reinforced with CFRP stirrups. The 0.6 in. (15.2 mm) diameter carbon fiber composite cables (CFCC) strands (produced by Tokyo Rope Mfg. Co., Ltd., Japan) were used to pretension two girders while the other two girders were pretensioned using 0.4 in. (10 mm) diameter indented Leadline bars (produced by Mitsubishi Chemical Corporation of Japan). Two of the four girders were reinforced for shear using 0.6 in. (15.2 mm) diameter CFCC stirrups and 0.4 × 0.2 in. (10 × 5 mm) Leadline bars of rectangular cross-section, while the two remaining beams were reinforced for shear using 0.6 in. (15 mm) diameter epoxy coated steel reinforcing bars. Anchorage systems were provided by the manufacturers of the reinforcement. In addition to the girders, a two-lane width of the deck slab was reinforced by 0.4 in. (10 mm) diameter indented Leadline bars similar to the reinforcement used for prestressing.
The 0.6 in. (15.2 mm) diameter C-bar (GFRP) reinforcement (produced by Marshall Industries Composites Inc. of the United States) has been used to reinforce the portion of the jersey-type barrier walls. Double-headed stainless steel tension bars of 0.75 in. (19 mm) diameter were used for the connection between the barrier wall and the deck slab. It should be noted that only a portion of the bridge was designed using FRP materials to ensure that the new materials be tested under the same conditions as conventional steel reinforcement.

The additional innovation on the Taylor Bridge consisted of fiber optic sensors that were installed on the CFRP, GFRP, and steel reinforcement to monitor it from a remote station. Thermocouples were used at different locations on the bridge to permit compensation for the temperature change. All data are down loaded via a telephone line to an engineer’s office. The Taylor Bridge is the winner of the prestigious Precast/Prestressed Concrete Institute’s Harry H. Edwards Award for Industry Advancement.

Crowchild Bridge- The Crowchild bridge (Rizkalla and Labossiere, 1999) is located in Calgary, Alberta. The original bridge was demolished in May 1997 and replaced by a system using steel girders, a steel-free concrete deck for the intermediate deck spans, and GFRP reinforcement for the cantilever sidewalks. The 0.6 in. (15 mm) diameter GFRP reinforcements, produced by Marshall Industries Composites Inc. of the United States, were used to reinforce the two cantilever sidewalks including the top reinforcement of the adjacent slabs. The composite action between the steel-free deck and the steel girder was achieved by using Nelson Studs welded to the top flange of the steel girder and the steel straps required for the arch action mechanism in the steel-free deck.

The Crowchild bridge is instrumented with 81 strain gages, 19 embedded gages, five thermisters, three smart glass reinforcing bars, and two fiber optic gages. A wireless data acquisition system, which consists of a 24-bit data acquisition chip, a radio transmitter, and a trigger is currently being developed to remotely monitor the bridge. A static truck load test and an ambient vibration test have been performed on the bridge and the preliminary results show extremely encouraging results. Calgary’s Crowchild Trail
Bridge was recognized by the Association of Consulting Engineers of Alberta with a Show Case Award for its innovative use of GFRP reinforcing bars a steel-free concrete bridge deck.

**Joffre Bridge** - The Joffre Bridge (Rizkall and Labossiere, 1999) is located over the St.-Fracois River in Sherbrooke, Quebec. It consists of a five-span [85 to 115 ft (26 to 35 m)] superstructure supported by steel girders spaced at 12 ft (3.7 m). Girders are designed in a composite action with the deck slab. Construction of the bridge started in August 1997 and the bridge opened to the traffic in December of the same year. The deck slabs of the bridge were reinforced with CFRP NEFMAC grids. The NEFMAC was produced by Autocon Composites Inc. of North York, Ontario.

The investigation presented in this report addresses the detailed evaluation of 7.5 and 10.5 mm diameter CFTC $1 \times 7$ stirrups [products of Diversified Composites, Inc. (DCI)] and CFCC $1 \times 7$ stirrups (provided by Tokyo Rope Mfg. Co. Ltd., Tokyo, Japan), and 9.5 mm diameter C-bar stirrups (products of Marshall Composite Industries Inc.) under uni-axial loading condition. A comprehensive study was conducted to explore the effects of embedded length on failure modes and strength of stirrups anchored into concrete blocks with discontinuous (type A) and continuous (type B) end anchorage configurations (Morphy et al., 1997). The discontinuous and continuous anchorage configurations simulate the performance of the stirrups with standard hooks and the performance of continuous stirrups in a beam, and govern the lower and upper bound on the uni-axial stirrup strengths, respectively. In addition, detailed shear and flexural response of box-beams reinforced with CFRP bars and CFRP stirrups and prestressed using seven bonded and six unbonded CFRP tendons are also discussed. The variation of various parameters such deflection and crack widths with applied loading, and the ultimate load capacity of beams tested in shear and flexure are presented. The effect of internal unbonded post-tensioning is also examined with regard to the ductility of the beam. Finally, shear and design examples, using the same dimensions and reinforcing details of the tested box-beams, are presented to show the design approach of the box-beams prestressed using bonded and unbonded tendons and reinforced with CFRP bars.
and stirrups. These examples demonstrate the agreement between the proposed design approach and the experimental results.
Three instrumented AASHTO I-beams of Structure A

Abutment

Back Wall

Deck Fascia and Concrete Barrier Wall

Typical Instrumentation of Shaded DT Beam:
30 - Internal Concrete Strain Gages
3 - External Displacement Transducer
4 - External Longitudinal Tendon Load Cells

Typical Instrumentation of Non-Shaded DT Beam:
4 - External Tendon Displacement Transducer

Six instrumented DT-beams of structure B shown shaded here

Three instrumented AASHTO I-beams of Structure A

Figure 2.1. Plan View of Bridge Street Bridge Project
Figure 2.2 Cross section of Bridge Street Bridge: Structure A has conventional AASHTO Type III girders and a new substructure; Structure B is the CFRP double tee bridge supported by an existing substructure
Figure 2.3. External longitudinal CFCC post-tensioning strands
CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Outline of the Experimental Program

The experimental program consisted of evaluation of (i) strength of CFRP stirrups, (ii) bond strength of CFRP bars and tendons, and (iii) shear and flexural responses of box-beams reinforced with CFRP stirrups and bars, and prestressed using bonded and unbonded CFRP tendons. One hundred eight specimens of rectangular shaped stirrups with two different end-anchorage configurations were evaluated. The first end-anchorage configuration, type A, consisted of standard hook end, while the second configuration, type B, consisted of a continuous end. Stirrups with 7.5, 9.5, and 10.5 mm (0.3, 0.374, and 0.41 in.) diameters were used. In addition, a total of nine box-beams, six for shear and three for flexure, were constructed, instrumented, and tested. Eight box-beams were prestressed using seven bonded pretensioning and six unbonded post-tensioning tendons. The remaining one box-beam was prestressed using only seven bonded pretensioning tendons, whereas the six unbonded post-tensioning tendons remained in the place without prestress. To assess the bond strength of CFRP bars and tendons, a total of twenty-five T-beams were fabricated, instrumented, and tested. The fabrication and construction details of stirrups, uni-axial stirrup test specimens, T-beams, and box-beams are presented in the following sections. The material properties of different types of CFCC and carbon fiber twisted cables (CFTC) strands (used in manufacturing of the test stirrups), and CFRP tendons/bars are presented in Tables 3.1 and 3.2, respectively, while Figures 3.1 and 3.2 show the different types of stirrups and CFRP bars/tendons used in the experimental program, respectively. The cross-section configuration of CFTC stirrup, provided by Diversified Composites, Inc. (DCI), was similar to that of CFCC stirrups.

3.2 Fabrication of CFTC Stirrups

Carbon fiber twisted cable (CFTC) stirrups were fabricated using CFTC $1 \times 7$ strands manufactured by Diversified Composites, Inc. (DCI). The CFTC $1 \times 7$ strands
consisted of seven CFTC 1 × 1 strands, each of which had a twist of one full revolution per foot length. Each strand (CFTC 1 × 1) was a collection of twelve ends of Torey M 40J carbon fiber. The mechanical properties of Torey M 40J carbon fiber and Toho Rayon carbon fiber (used for manufacturing CFTC 1 × 7 and CFCC 1 × 7 strands, respectively) are presented in Table 3.3.

The rectangular shape [762 × 203 mm (30 × 8 in.)] CFTC 1 × 7 stirrups were fabricated using CFTC 1 × 7 strands. Figure 3.3 shows the wrapping of CFTC 1 × 7 strand around a holding jig to give the initial form of the stirrup. The stirrup was secured against a tensioning plate with a threaded screw at one end of the holding jig. A torque wrench was used to extend the screw thereby putting the stirrup under a specific tension force before curing. Next, the entire assembly was cured in an oven at 190°F for 90 minutes. After curing, the stirrups were coated with an epoxy resin. The methods of the manufacturing of the CFCC stirrups, supplied by Tokyo Rope Mfg. Co. Ltd., and CFRP stirrups, supplied by Marshall Industries Composites, Inc.*, were proprietary and are not available.

3.3 Failure and Strength of Stirrups

In this section, details of the fabrication and construction of the test specimens for testing of the CFTC, CFCC, and CFRP stirrups are presented.

3.3.1 Construction Details

To simulate the performance of stirrups with a standard hook and continuous ends, all the CFRP stirrups were fixed (with an appropriate anchorage) into formwork as shown in Figure 3.4. For type A specimens, the anchored-end of the stirrups was debonded to simulate the performance of stirrups with a standard hook. For type B stirrups, the stirrups were debonded only at the continuous end. The debonding was facilitated by using plastic tubes to achieve various embedded lengths. These “debonding tubes” were secured into position with epoxy resin and duct tape. The debonding

* Marshall Industries Composites, Inc. is no longer in business.
Table 3.1. Average properties CFCC 1 × 7 strands, CFTC 1 × 7 strands, and steel rebar used as stirrups

<table>
<thead>
<tr>
<th>Type of bar</th>
<th>CFCC 1 × 7 strand</th>
<th>CFTC 1 × 7 strand</th>
<th>Steel rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturing Company</td>
<td>Tokyo Rope Mfg. Co. Ltd. (CFCC)</td>
<td>Diversified Composites, Inc. (DCI)</td>
<td></td>
</tr>
<tr>
<td>Nominal diameter, d, mm (in.)</td>
<td>7.5 (0.3), [10.5 (0.4)]*</td>
<td>7.5 (0.3), [10.5 (0.4)]*</td>
<td>9.5 (0.374)</td>
</tr>
<tr>
<td>Effective cross sectional area, mm$^2$ (in.$^2$)</td>
<td>30.4 (0.047), [55.7 (0.086)]</td>
<td>43.4 (0.067), [54.0 (0.084)]</td>
<td>70.9 (0.11)</td>
</tr>
<tr>
<td>Guaranteed strength, MPa (ksi)</td>
<td>1875 (272), [1867 (271)]</td>
<td>1156 (167.7), [1028 (148.9)]</td>
<td>414 (60)$^+$</td>
</tr>
<tr>
<td>Ultimate tensile strength, MPa (ksi)</td>
<td>1910 (277.0), [2280 (331)]</td>
<td>1218 (176.6), [1256 (182.0)]</td>
<td>1,689.3 (245)</td>
</tr>
<tr>
<td>Measured breaking force, kN (kips)</td>
<td>58.0 (13.0), [127 (28.5)]</td>
<td>52.8 (11.9), [67.7 (15.2)]</td>
<td>119.7 (26.9)</td>
</tr>
<tr>
<td>Elastic modulus, GPa (ksi)</td>
<td>137 (19,865), [137 (19,865)]</td>
<td>165 (23,925), [172 (24,940)]</td>
<td>200 (29,000)</td>
</tr>
<tr>
<td>Maximum % elongation</td>
<td>1.5 [1.5]</td>
<td>1.7 [1.1]</td>
<td>0.2$^{**}$</td>
</tr>
</tbody>
</table>

*Quantities within bracket [ ] refer to 10.5 mm diameter strands; $^+$Yield strength of steel.
$^{**}$Yield strain of steel
Table 3.2. Properties of CFRP stirrups and tendons

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Tendons</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DCI</td>
<td>Leadline™</td>
</tr>
<tr>
<td>Nominal diameter, d, mm (in.)</td>
<td>9.5 (0.374)</td>
<td>10 (0.394)</td>
</tr>
<tr>
<td>Cross sectional area, mm² (in.²)</td>
<td>70.9 (0.11)</td>
<td>78.7 (0.122)</td>
</tr>
<tr>
<td>Guaranteed strength, MPa (ksi)</td>
<td>1524 (221)</td>
<td>2262 (328)</td>
</tr>
<tr>
<td>Ultimate tensile strength, MPa (ksi)</td>
<td>1930 (280)</td>
<td>2862 (415)</td>
</tr>
<tr>
<td>Elastic modulus, GPa, ksi</td>
<td>131 (19,000)</td>
<td>147 (21,320)</td>
</tr>
<tr>
<td>Maximum elongation (%)</td>
<td>1.47</td>
<td>1.9</td>
</tr>
</tbody>
</table>

* Properties in axial direction; * Marshall Industries Composites Inc. is no longer in business.

Table 3.3. Mechanical properties of carbon fiber used in CFTC 1 × 7 and CFCC 1 × 7 stirrups

<table>
<thead>
<tr>
<th>Properties</th>
<th>Torey M 40J carbon fiber (CFTC 1 × 7)</th>
<th>Toho Rayon Carbon fiber (CFCC 1 × 7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength, MPa (ksi)</td>
<td>4410 (640)</td>
<td>3,630 (526)</td>
</tr>
<tr>
<td>Tensile modulus, GPa (ksi)</td>
<td>377 (54,677)</td>
<td>235 (34,083)</td>
</tr>
<tr>
<td>Percentage elongation at failure</td>
<td>1.2</td>
<td>1.5</td>
</tr>
</tbody>
</table>
configurations for anchor types A and B are shown in Figure 3.5. Concrete blocks, 250 × 273 × 200 mm (10 × 10.75 × 8 in.) in size, were cast by pouring concrete into each end block of the formwork (Figure 3.4). The concrete had a 28 days compressive strength of 48.3 MPa (7 ksi).

Each stirrup had a bend radius \((r_b)\) of 38 mm (1.5 in.). Three different embedded lengths, \(l_d\) (equal to \(r_b + d_b\), \(10d_b\), and \(15d_b\)) were considered for the debonding tube lengths. Here, \(d_b\) represents the nominal diameter of the strand/bar of stirrup. The free length of the stirrup between two concrete blocks was also kept constant at 292 mm (11.5 in.). It should be noted that when splitting of concrete rather than failure of stirrup led to the specimen failure, other formwork was fabricated with sufficient reinforcement (see Figure 3.6) to avoid failure of specimen due to concrete splitting.

### 3.3.2 Instrumentation Details

In this section, details of the various instrumentation devices used in the experimental setups of this investigation are outlined below. All the sensors were connected to an OPTIM Electronics MEGADAC 3415 data acquisition system equipped with sufficient number of channels. The channels of the data acquisition system accommodated all the sensors required for each test. A Pentium III computer equipped with test control software (TCS) package was used to manage and store the collected data. The computer was interfaced with the MEGADAC unit using an IEEE 488 card.

**Strain gages**

All the strain gages and associated supplementary materials were manufactured by Measurements Group Inc. These gages had a resistance of 350 ohms at a temperature of 24°C. The specified operating temperature range for strain gages was –100°C to 350°F (–75° to 175°C). The thermal expansion coefficient of gages used for measuring strains in the concrete and steel was \(6 \times 10^{-6}\), while the corresponding thermal expansion coefficients of gages used for measuring CFRP strains in CFRP was zero. Installation of
strain gages consisted of smoothening the surface of concrete, steel, and/or CFRP with a sand paper, degreasing using acidic solution, neutralizing with base solution, and then drying the corresponding bonding surface. Finally, strain gages were bonded in place using catalyst B200 and M-Bond 200 adhesive.

**Linear Motion Transducers**

Deflections were measured using linear motion transducers model P-50A. These linear motion transducers are commercially known as string-pots, which consists of retractable strings connected to a radial potentiometer. The resistance of the potentiometer is proportional to the distance to which the string is pulled giving a usable range of about 50 inches. To measure the deflection, an aluminum block was attached to the point of surface where deflection is desired, while the string-pot was mounted on a stationary surface. The string of the string-pot was attached to the non-movable aluminum block to measure the differential movement caused due to deflection between the aluminum block and the point of attachment of the string and concrete surface. These string pots were connected to the data acquisition system for collection of data.

**Linear Voltage Displacement Transducer (LVDT)**

Model number 1000-HCD LVDT manufactured by Schaevitz were used to measure the slippage of bar from the concrete. The LVDT consists of a threaded core passing through a coil assembly. The actual distance moved by the core is indicated by change in the voltage of LVDT system. The LVDT units have a double magnetic shielding, which makes them insensitive to any external influence. The 1000-HCD model provides voltage of ± 10 VDC, which offers a range between negative and positive one inch with a scale factor of 10.1714 volts dc/in.

**Load cells**

The load cells manufactured by PCB Piezotronics, Inc. were used to measure and monitor the pulling forces in the bonded pretensioning and unbonded post-tensioning tendons. The load cells had a maximum capacity of 166.9 kN (37.5 kips). The load cells
used for ultimate load carrying capacity of box-beams had a maximum capacity of 200 kips (890 kN). The maximum capacity of the load cell used for ultimate load test of T-beams was 85 kips (378 kN).

**DEMEC Points**

DEMEC points were attached to the concrete surface of the box-beams to measure concrete strain at different locations for transfer length measurement. Measurements were made manually using DEMEC gage ID-C112M with an accuracy of $5 \times 10^{-5}$ in. DEMEC stations, having a gage length of eight inches each, were used to measure strain at the middle of the span and both ends of the box-beams.

**3.3.3 Stirrup Test Setup**

Test setups for typical CFTC and CFRP stirrups with concrete blocks at their end are shown in Figures 3.7 and 3.8, respectively. Each stirrup was subjected to an axial tensile force by applying a relative displacement between the two concrete blocks using a 156 kN (35 kips) hydraulic jack. A 156 kN (35 kips) load cell was used to measure the applied force. An extensometer and a strain gage were used to measure strain in the stirrups. Steel plates were placed between the load cell and the concrete blocks to ensure a uniform load distribution on the concrete blocks. A similar test setup was employed for CFCC stirrups.

**3.4 T-Beam Bond Test**

A total of twenty-five T-beams were fabricated and tested to examine the bond characteristics of CFRP reinforcing bars/tendons with concrete. Nine T-beams were reinforced with DCI tendons; six T-beams were reinforced with CFRP bars, provided by Marshall Industries Composites, Inc. (MIC), and the remaining ten T-beams were reinforced with Leadline tendons provided by Mitsubishi Chemical Corporation, Japan. T-beams reinforced with a particular CFRP-bar were divided into groups of three different embedment lengths, i.e., thirty, twenty-four, and eighteen inches. In addition,
six T-beams reinforced with steel bars were also fabricated and tested in order to compare the bond characteristics of CFRP and steel bars. The embedment lengths for three

<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Number of specimens</th>
<th>Bar Material</th>
<th>Embedment Length, mm (in.)</th>
<th>Nominal Bar Diameter, d mm (in.)</th>
<th>(E_f/E_s)♣</th>
<th>f_{fu}♣ MPa (ksi)</th>
<th>f_{c}♣ MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TD-30</td>
<td>3</td>
<td>DCI</td>
<td>762 (30)</td>
<td>9.5 (0.374)</td>
<td>0.655</td>
<td>1930 (280)</td>
<td></td>
</tr>
<tr>
<td>TD-24</td>
<td>3</td>
<td>DCI</td>
<td>610 (24)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TD-18</td>
<td>3</td>
<td>DCI</td>
<td>457 (18)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TM-30</td>
<td>2</td>
<td>MIC</td>
<td>762 (30)</td>
<td>9.5 (0.374)</td>
<td>0.53</td>
<td>1896 (275)</td>
<td></td>
</tr>
<tr>
<td>TM-24</td>
<td>2</td>
<td>MIC</td>
<td>610 (24)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TM-18</td>
<td>2</td>
<td>MIC</td>
<td>457 (18)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TL-30</td>
<td>3</td>
<td>Leadline</td>
<td>762 (30)</td>
<td>10 (0.394)</td>
<td>0.735</td>
<td>2861 (415)</td>
<td></td>
</tr>
<tr>
<td>TL-24</td>
<td>3</td>
<td>Leadline</td>
<td>610 (24)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TL-18</td>
<td>4</td>
<td>Leadline</td>
<td>457 (18)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS-02</td>
<td>2</td>
<td>Steel</td>
<td>51 (2.0)</td>
<td>9.5 (0.374)</td>
<td>1.0</td>
<td>414* (60)</td>
<td></td>
</tr>
<tr>
<td>TS-1.5</td>
<td>2</td>
<td>Steel</td>
<td>38 (1.5)</td>
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<td></td>
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<tr>
<td>TS-01</td>
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<td>Steel</td>
<td>25 (1.0)</td>
<td></td>
<td></td>
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</table>

* Yield Strength; ♣ E_f, E_s, f_{fu}, and f_{c} represent the modulus of elasticity of CFRP bar and steel bar, ultimate strength of CFRP bar, and concrete strength, respectively.
different groups, each with two specimens, of steel reinforced T-beam specimens were two, one and half, and one inches. Details of T-beam bond specimens are presented in Table 3.4. In the designation of beams, letters T, D, M, L and S stand for T-beam, DCI reinforcing bar, MIC reinforcing bar, Leadline (MCC) bar, and steel bar, respectively.

The number associated with each T-beam designation represents the embedment length of the bars in inches. Figure 3.9 shows the configuration of T-beam bond specimens and their stirrups. As shown in Figure 3.9, the width and depth of the flange of the T-beams were 254 and 76 mm (10 and 3 in.), respectively. The total depth of each T-beam was 355.6 mm (14 in.), while the total and effective spans of T-beams were 1,829 and 1,575 mm (72 and 62 in.), respectively. Two top CFRP bars (Figure 3.9) of 1,778 mm (70 in.) length were used to hold the reinforcing stirrups, while the bottom reinforcing bar tested for bond strength was located at a distance of 38 mm (1.5 in.) from the bottom. The length of the bar tested for bond was 2,438.4 mm (8 ft). Each stirrup was 12 in. deep and 3.5 in. wide at the top and was tapered to 1.5 in. bend at the bottom (Figure 3.9). The radius of other bend in stirrups was 1.5 in.

3.4.1 Instrumentation of T-beams

Two linear-voltage displacement transducers (LVDT) were mounted on the protruding portions of the bar at both ends of the T-beam (Figure 3.10) to measure the slippage of the bar from the concrete. Threaded cores of each LVDT were attached to the end of the concrete beam. Two strain gages were installed on the bar at midspan, i.e., within the notched portion (Figure 3.10) of the T-beam to evaluate the tension force in the bar. A string-pot mounted on a fixed strut with its string connected to the top surface of the T-beam at midspan measured the midspan deflection.

3.4.2 Bond Test Setup and Testing

T-beams using CFRP bars were tested for bar/tendon bond characteristics using four-point load system as shown in Figure 3.11. The beam was simply supported at both
ends using hinge support at one end and a roller at the other end. The center of each support was located 127 mm (5 in.) from the corresponding beam end. A 100 × 100 × 12.5 mm (4 × 4 × 0.5 in.) steel box-section loading beam was used to transfer the applied central load to the T-beam. Beams were loaded statically to failure using MTS actuator and pump that had a maximum capacity of 365 kN (82 kips). A digital display unit connected to the actuator controlled the hydraulic pressure and the rate of loading.

3.5 Shear Test

In order to examine the response of the prestressed concrete box-beams under shear load, six box-beams were fabricated. Each beam was 4,877 mm (16 ft) long, 965 mm (38 in.) wide, and 305 mm (12 in.) deep. Each beam was heavily reinforced with double steel stirrups (at d/3) in one of the end one-third spans of the beam, while middle-third span of box-beam was reinforced with single steel stirrups at spacing three times that of heavily reinforced portion. The remaining one-third span of box-beams was reinforced with CFRP/steel stirrups at different spacings as per design. These arrangements of the stirrups were designed to cause shear failure in the region reinforced with CFRP stirrups. The box-beams, with different types and spacing of stirrups in one end-third span, consisted of one control beam (beam without shear reinforcement), one beam reinforced with 9.5 mm (0.374 in.) diameter steel stirrups, two beams reinforced with 9.5 mm (0.374 in.) diameter MIC CFRP stirrups, and two beams reinforced with 10.5 mm (0.41 in.) diameter CFCC stirrups. The center-to-center spacing of steel stirrups was d/2, while that of CFCC and MIC CFRP stirrups were d/2 in one beam and d/3 in the other beam. No beams was fabricated with CFTC stirrups. All the beams were reinforced longitudinally with four 9.5 mm (0.374 in.) diameter CFRP (DCI) bars in the bottom flange and seven 9.5 mm (0.374 in.) diameter CFRP (DCI) bars in the top flange. Prestressing was achieved using seven bonded pre-tensioning and six unbonded post-tensioning CFRP (DCI) tendons of 9.5 mm (0.374 in.) diameter. The effective prestressing force in each tendon was 92.5 kN (20.8 kips).
Table 3.5 Details of tested box-beams

<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Stirrups</th>
<th>Prestressing Tendons</th>
<th>$E_{ve}/E_s$</th>
<th>$f_u$# MPa (ksi)</th>
<th>Spacing##</th>
<th>$f_c'$ MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2</td>
<td>MIC</td>
<td>7 pre-tensioning</td>
<td>0.53</td>
<td>1896 (275)</td>
<td>d/2</td>
<td>48 (7)</td>
</tr>
<tr>
<td>M3</td>
<td></td>
<td>and 6 Post-tensioning</td>
<td></td>
<td></td>
<td>d/3</td>
<td></td>
</tr>
<tr>
<td>T2</td>
<td>CFCC</td>
<td></td>
<td>0.67</td>
<td>1868 (271)</td>
<td>d/2</td>
<td></td>
</tr>
<tr>
<td>T3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>d/3</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>Steel</td>
<td></td>
<td>1.0</td>
<td>414 (60)*</td>
<td>d/2</td>
<td></td>
</tr>
<tr>
<td>N0</td>
<td>--------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Yield Strength,* $d$ is the effective beam depth = 254 mm (10 in.); $f_u$ is the ultimate strength of bar/tendon; $E_{ve}$ is the Young’s modulus of CFRP stirrups; and $E_s$ is the Young’s modulus of steel stirrups.
Details of the box-beams are given in Table 3.5. Depending upon the type and spacing of stirrups in one of the two end one-third spans, box-beams were designated by different nomenclature. The beams reinforced with CFRP (MIC) stirrups at a spacing of \(d/2\) and \(d/3\) are designated as M2 and M3, respectively, while those reinforced with CFCC stirrups are designated as T2 and T3, respectively. Similarly, the beam reinforced with steel stirrups at a spacing of \(d/2\) is designated as S2 and the beam without shear reinforcement is designated as N0.

All six box-beams had a hollow rectangular cross-section with two cells throughout the entire span except at the ends. Each cell was 305 mm (12 in.) wide and 102 mm (4 in.) deep. Two solid blocks of length 305 mm (1 ft) each, were provided to allow smooth transfer of the prestressing force at the live end of the beam. Figure 3.12 shows the cross-sectional details of the box-beam. Figures 3.13 to 3.18 show the arrangement of the stirrups in beams M2, M3, T2, T3, S2, and N0, respectively.

### 3.5.1 Prestressing

As mentioned earlier, all the six box-beams were prestressed using seven pretensioning and six post-tensioning CFRP (DCI) tendons. Each tendon was stressed to an average of 92.5 kN (20.8 kips). The prestressing system consisted of a long stroke center-hole jack, a hydraulic pump with pressure gage, and prestressing chair. Jacking force was monitored using pressure of the hydraulic pump and the load cells readings. Strain gages were installed on the prestressing tendons to observe the change in the strain during the prestressing process. Four bulkheads fixed to the floor were used in the pretensioning of the tendons allowing the fabrication of two beams at a time, each of the two pairs of abutments were spaced 6,858 mm (270 in.) apart.

### 3.5.2 Anchorage System

The CFRP prestressing tendons were provided with a special anchorage system at each end to facilitate pulling the tendons without damaging their ends. This anchorage system consisted of the following:
a) Aluminum tube: A 152 mm (6 in.) tube made of a special aluminum alloy, having an outer diameter of 12.7 mm (0.5 in.) and a thickness of 1.6 mm (1/16 in.).

b) Sleeve: A cylinder threaded from the outside with an outer diameter of 50.8 mm (2 in.) and a length of 152 mm (6 in.). The inside groove was conical in shape in order to wedge the CFRP tendons inside.

c) Wedge: A conical piece of metal having its outer dimensions set to fit inside the sleeve. The wedge was split into two halves with an inner groove to accommodate the aluminum tube; the inner groove was indented to provide firm grip on the tube.

The CFRP tendons were degreased with acetone for a length of 152 mm (6 in.) from the end and inserted in the aluminum tube, which was in turn fitted in the groove between the two halves of the wedge. A tapered piece of Teflon tape was wrapped around the wedge before inserting it in the sleeve. The wedge was then pushed into the sleeve using a pre-pushing system, consisting of a jack and a hydraulic pump. The anchorage system was set between the jack and a stationary plate and pressed with a force of 178 kN (40 kips). Figure 3.19 shows the components of the anchorage system, while Figure 3.20 shows the crimping system of CFRP tendons.

3.5.3 Jacking Process

The tendons were anchored from one end using a lock nut and pulled from the other end of the box-beam form. The sleeve was connected to a threaded steel rod using a coupler; the steel rod was then pulled using a center-hole hydraulic jack. The jack was pushing against a prestressing chair, which was in turn bearing on the bulkheads and concrete in pretensioning and post-tensioning, respectively. When the pulling force on the tendon reached the required value, the elongation of the tendon was measured. Spacers were placed between the bulk head/beam and the lock nut, and the lock nut was tightened to maintain the prestressing force in the tendon.
Table 3.6  Prestressing forces for box-beams used for shear test

<table>
<thead>
<tr>
<th>Beam Designation</th>
<th>Tendon Number</th>
<th>Pre-stressing force (kN (kips))</th>
<th>Prestressing stress (MPa (ksi))</th>
<th>Elongation (mm (in.))</th>
<th>End Slip (mm (in.))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Pre-stressing force</td>
<td>Prestressing stress</td>
<td>Elongation</td>
<td>Dead End</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(kN (kips))</td>
<td>(MPa (ksi))</td>
<td>mm (in.)</td>
<td></td>
</tr>
<tr>
<td>M2</td>
<td>1</td>
<td>93.5 (21)</td>
<td>1193 (173)</td>
<td>61 (2.4)</td>
<td>3.0 (0.12)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>97.9 (22)</td>
<td>1241 (180)</td>
<td>58 (2.3)</td>
<td>0.3 (0.01)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>89.0 (20)</td>
<td>1131 (164)</td>
<td>61 (2.4)</td>
<td>0.8 (0.03)</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>89.0 (20)</td>
<td>1131 (164)</td>
<td>58 (2.3)</td>
<td>1.3 (0.05)</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>93.5 (21)</td>
<td>1186 (172)</td>
<td>58 (2.3)</td>
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<td></td>
<td>6</td>
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<td>1186 (172)</td>
<td>58 (2.3)</td>
<td>4.6 (0.18)</td>
</tr>
<tr>
<td></td>
<td>7</td>
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<td>58 (2.3)</td>
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<tr>
<td>M3</td>
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<tr>
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<td>64 (2.5)</td>
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<tr>
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<td>58 (2.3)</td>
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<tr>
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<tr>
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<td>4.6 (0.18)</td>
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Table 3.6 Contd.

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<th>Beam Designation</th>
<th>Tendon Number</th>
<th>Prestressing force kN (kips)</th>
<th>Prestressing Stress MPa (ksi)</th>
<th>Elongation mm (in.)</th>
<th>End Slip mm (in.)</th>
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<td>Dead End</td>
<td>Live End</td>
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<td>61 (2.4)</td>
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<td></td>
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<td>1186 (172)</td>
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<td>64 (2.5)</td>
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</table>
Figures 3.21 to 3.24 show the different stages of prestressing, and Table 3.6 shows the details of prestressing forces applied to each tendon. The number designating each tendon represents the corresponding CFRP pretensioning tendon (Figure 3.12). It should be noted that average prestressing force in each pretensioning CFRP tendon was about 57% of the guaranteed strength of DCI tendons.

### 3.5.4 Beam Instrumentation

To obtain the stirrup strain, electrical resistance strain gages were fixed on the stirrups. These strain gages were protected with water resistant polyurethane coating and a PVC tube placed around it. Each beam was also instrumented with fifteen strain gages attached on the concrete surface at midspan. Five strain gages were provided at the top surface, and five on each side of the beam. String-pots provided at the midspan and two quarter-span points monitored the corresponding deflections. To measure slippage of tendon caused due to the release of prestressing force, three aluminum rings were fixed on each tendon. Distances between the three rings and the face of the concrete were measured before and after the release of each tendon using vernier calipers. Figure 3.25 shows the release of prestressing force in a tendon and the aluminum rings fixed to pretensioning tendons. Load cells, installed at the dead end of box-beams, were used to measure the pretensioning forces in each tendon. Details of tendon prestressing forces and stresses, elongations, and end slips for each beam are presented in Table 3.6.

Similar to bonded pretensioning tendons, load cells were used to monitor the post-tensioning forces in unbonded tendons during and after the initial post-tensioning. The strain gages glued to the stirrups prior to cast of concrete together with additional strain gages mounted on the concrete monitored and recorded the vertical and horizontal strains at desired locations. Five DEMEC rosettes were fixed on each side of the beam to monitor propagation of shear cracks in shear critical zones. Figure 3.26 shows the arrangements of DEMEC points to measure the width of shear crack.
3.5.5 Test Setup

The beams were simply supported on two rollers 4,572 mm (15 ft) apart. Each roller had a length equal to the width of the beams and was resting on a steel support. A jack with an attached load cell and a hydraulic pump, each having a capacity of 890 kN (200 kips) were used in the test. As shown in Figure 3.27, the beams were loaded over the entire width using a four-point loading system. Steel plates lined with rubber sheets were used at the four points of loading to distribute the load evenly throughout the width of the beam.

All sensors such as strain gages and string-pots were connected to the data acquisition system, which was used to monitor all the reading throughout the entire testing. The beams were first loaded to a load of 178 kN (40 kips), which was less than the cracking load, and then unloaded. Two more loading cycles were applied to the beams before loading to failure. DEMEC rosette readings were taken at 89 kN (20 kips) increments. The measured DEMEC data were used later to compute the width of shear cracks.

3.6 Measurements of Transfer Lengths

In addition to the six-beams tested for shear, three similar box-beams were constructed and instrumented for transfer length measurements and flexure test. Transfer lengths were measured using strain measurements obtained from readings of DEMEC points for all nine box-beams. To measure the transfer length, each box-beam was provided with a total of four sets of DEMEC stations placed at the level of the pretensioning tendons at each side of the both ends of the beam (Figure 3.28). The two longitudinal reinforcing bars and the seven bonded pretensioning tendons were provided with strain gages at midspan. In addition, unbonded post-tensioning tendons were equipped with load cells to monitor the effect of beam loading on the post-tensioning force. Additional strain gages were mounted on the concrete to monitor and record the horizontal strains at desired locations. To predict the decompression load necessary to estimate the effective prestress in pretensioning tendons, four strain gages were installed beside the first initiated crack as shown in Figure 3.29.
3.7 Flexure Test Setup

The three box-beams fabricated for flexure were tested under a similar four-point loading system (Figure 3.30) as for the shear except that the length of loading beam was different than that for shear test. The length of loading beam in shear test was 1,422 mm (56 in.), while it was 508 mm (20 in.) for flexure test. This loading arrangement created the maximum moment at the midspan. Two of the box-beams tested for flexure were prestressed using seven bonded and six unbonded post-tensioning tendons. The third box-beam tested for flexure was prestressed using seven bonded pretensioning tendons without post-tensioning forces in the unbonded tendons. The unbonded tendons were installed with anchor heads at both ends of the beam. The instrumentation and testing procedure for the flexure beam were the same as that for the shear test. Details of the test specimens are given in Table 3.7, while prestressing details are given in Table 3.8. In the designation of tested box-beams, the first letter (D or L) represents the type of tendon/bar used for prestressing/reinforcing the beam, while the second letter (P or N) represents whether the beam had post-tensioned forces. The first letter, D refers to the CFRP (DCI) tendons/bars while L refers to CFRP Leadline (MCC) tendons/bars. It should be noted that each tendon was prestressed to a load of about 89 kN (20 kips). The cross-section details of the beams tested in flexure are the same as for the beams tested in shear except that the two bottom non-prestressing tendons were removed (see Figure 3.31) from the flexural box-beam section. The center-to-center distance between the stirrups was kept as d/3, i.e., 76 mm (3 in.) to avoid premature shear failure. The longitudinal reinforcement details for the box-beam are shown in Figure 3.31. All the three box-beams tested in flexure were subjected to loading and unloading cycles before the ultimate loading. The loading and unloading sequence applied on the box-beams before ultimate loading was necessary to evaluate the inelastic energy absorbed in the box-beams and energy ratio as a measure of ductility index of the CFRP reinforced and prestressed concrete box-beams. Finally, all the three box-beams were statically loaded to failure.
Table 3.7 Details of box-beams tested for flexure

<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Flexural Reinforcement</th>
<th>Pre-tension</th>
<th>Post-tension</th>
<th>(E_f / E_s)*</th>
<th>f_{fu}* (MPa (ksi))</th>
<th>f_{c'}* (MPa (ksi))</th>
</tr>
</thead>
<tbody>
<tr>
<td>DP1</td>
<td>DCI</td>
<td>7 DCI tendons</td>
<td>6 DCI tendons</td>
<td>0.655</td>
<td>1,930 (280)</td>
<td></td>
</tr>
<tr>
<td>DN2</td>
<td>DCI</td>
<td>7 DCI tendons</td>
<td>6 DCI tendons without post-tensioning force</td>
<td>0.655</td>
<td>1,930 (280)</td>
<td>48 (7)</td>
</tr>
<tr>
<td>LP3</td>
<td>Leadline</td>
<td>7 Leadline tendons</td>
<td>6 Leadline tendons</td>
<td>0.735</td>
<td>2,861 (415)</td>
<td></td>
</tr>
</tbody>
</table>

E_f refers to Young’s modulus of elasticity of CFRP tendons/bars; E_s refers to Young’s modulus of elasticity of steel; f_{fu} refers to specified strength of CFRP tendons/bars; and f_{c'} refers to the 28-day strength of concrete.
<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Tendon Number</th>
<th>Prestressing force kN (kips)</th>
<th>Stress MPa, (ksi)</th>
<th>Elongation, mm (in.)</th>
<th>End Slip mm (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Dead End</td>
</tr>
<tr>
<td>DP1</td>
<td>1</td>
<td>91.2 (20.5)</td>
<td>1,285 (186.4)</td>
<td>61 (2.4)</td>
<td>0.8 (0.03)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>97.9 (22.0)</td>
<td>1,379 (200.0)</td>
<td>58 (2.3)</td>
<td>1.3 (0.05)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>93.5 (21.0)</td>
<td>1,316 (190.9)</td>
<td>71 (2.8)</td>
<td>1.0 (0.04)</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>89.0 (20.0)</td>
<td>1,254 (181.8)</td>
<td>58 (2.3)</td>
<td>2.5 (0.10)</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>95.7 (21.5)</td>
<td>1,348 (195.5)</td>
<td>64 (2.5)</td>
<td>1.5 (0.06)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>93.5 (21.0)</td>
<td>1,316 (190.9)</td>
<td>61 (2.4)</td>
<td>3.0 (0.12)</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>95.7 (21.5)</td>
<td>1,348 (195.5)</td>
<td>58 (2.3)</td>
<td>4.6 (0.18)</td>
</tr>
<tr>
<td>DN2</td>
<td>1</td>
<td>95.7 (21.5)</td>
<td>1,205 (174.8)</td>
<td>61 (2.4)</td>
<td>2.0 (0.08)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>93.5 (21.0)</td>
<td>1,177 (170.7)</td>
<td>61 (2.4)</td>
<td>0.8 (0.03)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>95.7 (21.5)</td>
<td>1,205 (174.8)</td>
<td>61 (2.4)</td>
<td>0.8 (0.03)</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>97.9 (22.0)</td>
<td>1,234 (178.9)</td>
<td>58 (2.3)</td>
<td>1.3 (0.05)</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>93.5 (21.0)</td>
<td>1,177 (170.7)</td>
<td>58 (2.3)</td>
<td>1.0 (0.04)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>95.7 (21.5)</td>
<td>1,205 (174.8)</td>
<td>58 (2.3)</td>
<td>4.6 (0.18)</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>89.0 (20.5)</td>
<td>1,149 (166.7)</td>
<td>58 (2.3)</td>
<td>2.5 (0.10)</td>
</tr>
<tr>
<td>LP3</td>
<td>1</td>
<td>97.9 (22.0)</td>
<td>1,379 (200.0)</td>
<td>61 (2.4)</td>
<td>0.8 (0.03)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>89.0 (20.5)</td>
<td>1,285 (186.4)</td>
<td>61 (2.4)</td>
<td>1.8 (0.07)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>97.9 (22.0)</td>
<td>1,379 (200.0)</td>
<td>56 (2.2)</td>
<td>2.3 (0.09)</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>89.0 (20.0)</td>
<td>1,254 (181.8)</td>
<td>61 (2.4)</td>
<td>2.0 (0.08)</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>93.5 (21.0)</td>
<td>1,316 (190.9)</td>
<td>64 (2.5)</td>
<td>1.5 (0.06)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>95.7 (21.5)</td>
<td>1,348 (195.5)</td>
<td>58 (2.3)</td>
<td>4.6 (0.18)</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>89.0 (20.0)</td>
<td>1,254 (181.8)</td>
<td>58 (2.3)</td>
<td>0.8 (0.03)</td>
</tr>
</tbody>
</table>
Figure 3.1 Different types of stirrups
Figure 3.2 Different types of bars and tendons
Figure 3.3 Jigs for holding CFTC $1 \times 7$ rectangular-shaped stirrup
Figure 3.4  Arrangement of stirrups in formwork
Figure 3.5 Details of specimen types A and B (Morphy et al., 1997)
Figure 3.6  Reinforcement details (within formwork) to avoid concrete splitting

Figure 3.7  Test setup for 10.5 mm diameter CFTC stirrups (provided by DCI)
Figure 3.8 Test setup for 9.5 mm diameter CFRP stirrups (provided by MIC)
**Figure 3.9** Configuration of T-beam bond test specimens
Figure 3.10 Instrumentation of T-beam bond test
Figure 3.11  Setup of T-beam bond test
Figure 3.12 Cross-sectional details of box-beams
Figure 3.15  Reinforcement details of Beam T2

Figure 3.16  Reinforcement details of Beam T3
Figure 3.17  Reinforcement details for Beam S2

Figure 3.18  Reinforcement details of Beam N0

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Figure 3.19 Components of the anchorage system
Figure 3.20 Crimping system of CFRP tendons
Figure 3.21 Prestressing of tendons at live end of the box-beam
Figure 3.22 Measurement of elongation of tendon at live end of the box-beam
Figure 3.23 Placement of spacer between abutment and lock nut after the prestressing force is applied
Figure 3.24  Anchorage assembly after pretensioning of tendons
Figure 3.25  Release of prestressing tendons
Figure 3.26  Arrangement of DEMEC points in shear-span to measure crack width
Figure 3.27 Shear test setup of tested box-beam
Figure 3.28  Arrangement of DEMEC points at the beam end to measure transfer length
Figure 3.29  Strain gages fixed adjacent to the first initiated crack at bottom of box-beam to measure decompression load
Figure 3.30  Four-point flexure load setup
* Beam DN2 has no post-tensioning force on the tendons

Figure 3.31 Reinforcement details of box beams tested for flexure
CHAPTER 4
RESULTS AND DISCUSSION

4.1 Introduction

In this chapter, measured CFRP stirrup strengths, bond strength of CFRP bars, transfer lengths of CFRP tendons, and shear and flexural responses of box-beams are presented and discussed. Based on the experimental results, various empirical strength models for different types of stirrups are recommended. These strength models for CFRP stirrups were developed from the test results of 7.5 and 10 mm (0.3 and 0.4 in.) diameter CFTC and CFCC strands, and 9.5 mm (0.374 in.) diameter MIC CFRP bars. In addition, based on beam tests, an effective stirrup stress model is suggested to predict the actual effective stress developed in the CFRP stirrups at ultimate failure of box-beams. The effective stress model is useful in computing the contribution of CFRP stirrups to shear strength of beams. The effect on the stirrup strength of various parameters such as embedment lengths, anchor types, and tail lengths is discussed in detail.

The bond characteristics of CFRP bars/tendons were studied by evaluating the bond strength and load-slip relations for DCI and Leadline (MCC) tendons, and CFRP (MIC) bars using T-beam bond test specimens. Similarly, transfer lengths of the DCI and Leadline tendons were measured from the concrete strain distribution of CFRP prestressed concrete box-beams. The transfer lengths were compared with those obtained from available empirical models. In addition, the shear and flexural responses of prestressed box-beams using bonded pretensioning and unbonded post-tensioning DCI and Leadline tendons and reinforced with MIC CFRP bars and MIC CFRP stirrups are presented and discussed in detail. The shear and flexural response characteristics consist of evaluation of shear and flexural strengths, stiffness, deflections, strains, crack widths, and energy ratios for the tested prestressed box-beams. The experimental results for stirrups, bond characteristics, transfer lengths, and shear and flexural responses of box-beams are presented below in different sections.
4.2 Stirrups

Figure 4.1 shows typical stress-strain relationships for 7.5 mm (0.3 in.) CFTC $1 \times 7$ and CFCC $1 \times 7$ stirrups of anchorage type A, tail length\(*) of $11d_b$, and embedded length of $15d_b$. It can be seen that the CFTC $1 \times 7$ stirrups had a higher elastic modulus than the CFCC $1 \times 7$ stirrups but lower uni-axial strength. The stirrup response is linearly elastic up to failure; however, the ultimate stirrup strength is lower than the ultimate strand strength (Table 3.1). This is attributed to the bend effect in stirrups. The failure stress and failure modes of 7.5 and 10.5 mm (0.3 and 0.41 in.) diameter CFTC $1 \times 7$ stirrups for different embedded lengths and anchorage types are presented in Table 4.1. The tail lengths of CFTC $1 \times 7$ stirrups were 11 times the nominal diameter ($d_b$). In this table, it is also shown that CFTC $1 \times 7$ stirrups failed primarily by strand rupture at the end of the debonded length inside the concrete block or at the bend.

In general, for a particular embedded length, stirrups with anchorage type B (debonded at the continuous end) show greater load carrying capacity than those with anchor type A (debonded on the standard hook end side). Also, as expected for a particular anchorage type, a higher load carrying capacity was observed for the specimens with longer embedded lengths. Note that the ratios of failure stress to guaranteed strength of 10.5 mm (0.41 in.) diameter strands are lower than those for 7.5 mm (0.3 in.) diameter stirrups. It is also worth noting that a decrease in the embedded length of a stirrup increased the possibility of failure at the bend zone and/or at the end of debonded length inside the concrete block, and resulted in a reduction of stirrup strength. These reductions in strength for 7.5 mm (0.3 in.) diameter CFTC $1 \times 7$ stirrups were (due to bend effect) 40 percent and 20 percent for anchorage types A and B, respectively. The reduction in strength of 10.5 mm (0.41 in.) diameter CFTC $1 \times 7$ was 65 percent for both anchorage types.

\*) Tail length is defined as the length of the straight end portion of the stirrup forming the standard hook.
<table>
<thead>
<tr>
<th>Material type</th>
<th>Nominal diameter, (d_b) mm (in.)</th>
<th>Stirrup anchorage type</th>
<th>Embedded length, (l_d) mm (in.)</th>
<th>Failure stress, (f_u) MPa (ksi)</th>
<th>((f_u/f_{tg})^\dagger)</th>
<th>Typical failure mode **</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFTC 1 × 7 (11(d_b))^*</td>
<td>7.5(0.3)</td>
<td>A</td>
<td>46 (1.8)</td>
<td>746 (108)</td>
<td>0.64</td>
<td>M</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>75 (3.0)^††</td>
<td>836 (121)</td>
<td>0.72</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>113 (4.4)^†††</td>
<td>1030 (149)</td>
<td>0.89</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B</td>
<td>46 (1.8)</td>
<td>942 (137)</td>
<td>0.81</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>75 (3.0)</td>
<td>1143 (166)</td>
<td>0.99</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>113 (4.4)</td>
<td>1263 (183)</td>
<td>1.09</td>
<td>E</td>
</tr>
<tr>
<td>CFTC 1 × 7 (11(d_b))^*</td>
<td>10.5 (0.4)</td>
<td>A</td>
<td>49 (1.9)</td>
<td>391 (57)</td>
<td>0.38</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>105 (4.1)^††</td>
<td>639 (93)</td>
<td>0.62</td>
<td>M, B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>158 (6.2)^†††</td>
<td>731 (106)</td>
<td>0.71</td>
<td>M, B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B</td>
<td>49 (1.9)</td>
<td>389 (56)</td>
<td>0.38</td>
<td>S, B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>105 (4.1)</td>
<td>711 (103)</td>
<td>0.69</td>
<td>B, M</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>158 (6.2)</td>
<td>929 (135)</td>
<td>0.90</td>
<td>M</td>
</tr>
</tbody>
</table>

The value refers to tail length of a stirrup; ** failure modes: M- rupture occurred between concrete blocks; E-rupture occurred at end of debonding inside block; B-rupture occurred at bend; S-excessive slippage of stirrup strands without stirrup failure. ^\dagger f_{tg} denotes guaranteed strength of stirrup strands; † \(l_d = r_b + d_b\); †† \(l_d = 10d_b\); ††† \(l_d = 15d_b\); \(f_u\) and \(f_{tg}\) are axial stirrup strength and guaranteed strength of stirrup strand, respectively.
Table 4.2 Results for CFCC $1 \times 7$ and C-bar (MIC) stirrups

<table>
<thead>
<tr>
<th>Material type</th>
<th>Nominal diameter, $d_b$ mm (in.)</th>
<th>Stirrup anchorage type</th>
<th>Embedded length, $l_d$ mm (in.)</th>
<th>Failure stress, $f_u$ MPa</th>
<th>$(f_u/f_{tg})^\psi$</th>
<th>Typical failure mode**</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFCC $1 \times 7$ (11$d_b$)*</td>
<td>7.5 (0.3)</td>
<td>A</td>
<td>46 (1.8)$^\dagger$</td>
<td>1230 (178)</td>
<td>0.66</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>75 (3.0)$^{++}$</td>
<td>1496 (217)</td>
<td>0.80</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>113 (4.4)$^{+++}$</td>
<td>1846 (268)</td>
<td>0.98</td>
<td>E, M</td>
</tr>
<tr>
<td></td>
<td>75 (3.0)</td>
<td>B</td>
<td>1201 (174)</td>
<td>0.64</td>
<td>S, M</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1797 (261)</td>
<td>0.96</td>
<td>S, M</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>1898 (275)</td>
<td>1.01</td>
<td>S, M</td>
<td></td>
</tr>
<tr>
<td>C-bar (MIC) (12 $d_b$)*</td>
<td>9.5 (0.374)</td>
<td>A</td>
<td>48 (1.9)$^\dagger$</td>
<td>540 (78)</td>
<td>0.29</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>79 (3.1)$^{++}$</td>
<td>689 (100)</td>
<td>0.37</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>119 (4.7)$^{+++}$</td>
<td>856 (124)</td>
<td>0.46</td>
<td>M</td>
</tr>
<tr>
<td></td>
<td>79 (3.1)$^{++}$</td>
<td>B</td>
<td>670 (97)</td>
<td>0.36</td>
<td>M</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>838 (122)</td>
<td>0.45</td>
<td>E</td>
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</tr>
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<td></td>
<td></td>
<td>968 (140)</td>
<td>0.52</td>
<td>E</td>
<td></td>
</tr>
</tbody>
</table>

* The value refers to the tail length of stirrups; ** failure modes: M-rupture occurred between concrete blocks; E-rupture occurred at end of debonding inside block; B-rupture occurred at bend; S-excessive slippage of stirrup strands without stirrup failure; $^\psi f_{tg}$ denotes guaranteed strength of stirrup strands; $^\dagger l_d = r_b + d_b$; $^{++} l_d = 10d$; $^{+++} l_d = 15d$; $f_u$ and $f_{tg}$ are the axial stirrup strength and guaranteed strength of stirrup strand/bar, respectively.
The failure stresses and failure modes for 7.5 mm (0.3 in.) diameter CFCC $1 \times 7$ stirrups and 9.5 mm (0.374 in.) diameter C-bar stirrups are presented in Table 4.2. It was observed that the 7.5 mm (0.3 in.) diameter CFCC $1 \times 7$ stirrups failed primarily by strand breaking at the bend or in between the concrete blocks. The C-bar (MIC) stirrups also exhibited similar failure characteristics as the CFTC and CFCC stirrups.

It should be noted that 10.5 mm (0.41 in.) diameter CFCC $1 \times 7$ stirrups did not rupture due to premature splitting in the end concrete blocks, and the strengths of these stirrups did not reach the guaranteed strength of the strands. Therefore, their test results are not presented because they do not represent the actual strength of stirrups. This is attributed to the high strength of CFCC strands. For 7.5 mm (0.3 in.) diameter stirrups, the ratio of failure stress to the guaranteed strength was higher for the CFTC $1 \times 7$ stirrups than for the CFCC $1 \times 7$ stirrups, regardless of anchorage type. Also, the bend effect, as in the case of CFTC $1 \times 7$ stirrups, resulted in reduction of stirrup strength by 40 percent (same as reported by Morphy et al., 1997) and 50 percent for 0.3 and 0.4 in. (7.5 and 10.5 mm) diameter CFCC $1 \times 7$ stirrups, respectively. The corresponding reduction in the strength for the C-bar (MIC) stirrups was 72 percent.

Figures 4.2 and 4.3 show the relationships between the ratio of the failure stress to guaranteed strength and the ratio of embedded length to effective diameter (Tokyo Rope Mfg. Co. Ltd., 1993) for 7.5 mm (0.3 in.) diameter CFCC $1 \times 7$ and CFTC $1 \times 7$ stirrups, respectively. In these figures, $d_e$ refers to the effective diameter of the stirrup strand, and is calculated using the effective cross-sectional area* of the strand. The higher strengths were observed for anchorage type B (continuous) in comparison to anchorage type A (standard hook). Note that the strength of 7.5 mm (0.3 in.) diameter CFCC $1 \times 7$ stirrups with anchorage type B (Figure 4.2) reached the guaranteed value for embedded lengths 16.3 times the effective diameter of the strand, while 7.5 mm (0.3 in.) diameter CFTC $1 \times 7$ stirrups with anchorage type B (Figure 4.3) reached the guaranteed value for embedded lengths 11.8 times the effective diameter. However, 7.5 mm (0.3 in.) diameter CFCC $1 \times 7$

---

* The effective cross-sectional area is calculated by dividing the weight of a strand of unit length by the specific gravity of the strand material.
7 and CFTC 1 × 7 stirrups with anchorage type A did not reach the guaranteed strength even for embedded lengths of 18.3 and 15.1 times the effective diameter, respectively. This is attributed to the premature failure of the stirrups due to a higher stress concentration at the end of the debonding tube and/or at the bend. The following relationships for predicting the strength of 7.5 mm (0.3 in.) diameter CFCC 1 × 7 and CFTC 1 × 7 stirrups with 38 mm (1.5 in.) bend radius for various embedded lengths have been established:

**7.5 mm (0.3 in.) diameter CFCC 1 × 7 stirrups with tail length 11d₀**

**Anchor type A**

\[
\frac{f_u}{f_{tg}} = 0.6
\]

for \( \frac{l_d}{d_e} < 7.4 \)  \hspace{1cm} (4.1a)

\[
= 0.0297 \left( \frac{l_d}{d_e} \right) + 0.44
\]

for \( 7.4 \leq \frac{l_d}{d_e} \leq 18.9 \) \hspace{1cm} (4.1b)

\[
= 1.0
\]

for \( \frac{l_d}{d_e} > 18.9 \) \hspace{1cm} (4.1c)

**Anchor type B**

\[
\frac{f_u}{f_{tg}} = 0.6
\]

for \( \frac{l_d}{d_e} < 7.4 \) \hspace{1cm} (4.2a)

\[
= 0.0331 \left( \frac{l_d}{d_e} \right) + 0.46
\]

for \( 7.4 \leq \frac{l_d}{d_e} \leq 16.3 \) \hspace{1cm} (4.2b)

\[
= 1.0
\]

for \( \frac{l_d}{d_e} > 16.3 \) \hspace{1cm} (4.2c)

**7.5 mm (0.3 in.) diameter CFTC 1 × 7 stirrups with tail length 11d₀**

**Anchor type A**

\[
\frac{f_u}{f_{tg}} = 0.6
\]

for \( \frac{l_d}{d_e} < 6.1 \) \hspace{1cm} (4.3a)
\[ \frac{f_{u}}{f_{ug}} = 0.8 \quad \text{for } \frac{l_{d}}{d_{e}} < 6.1 \quad (4.4a) \]

\[ = 0.0306 \left( \frac{l_{d}}{d_{e}} \right) + 0.64 \quad \text{for } 6.1 \leq \frac{l_{d}}{d_{e}} \leq 11.8 \quad (4.4b) \]

\[ = 1.0 \quad \text{for } \frac{l_{d}}{d_{e}} > 11.8 \quad (4.4c) \]

Anchor type B

The above design equations for anchor type A and anchor type B give the lower and upper bounds of uni-axial stirrup strength for a specific embedded length. The lower limit of the uni-axial stirrup strength can be used to limit the effective strength of stirrups. The effective strength of stirrups is used to evaluate the shear strength contribution of the stirrups used in reinforced/prestressed concrete beams (Fam et al., 1995, and ACI 440, 2000).

By comparing the uni-axial strength models of 7.5 mm (0.3 in.) diameter CFCC 1 \( \times \) 7 stirrups with those of Morphy et al. (1997) for \( \frac{l_{d}}{d_{e}} = 10 \), it is observed that the failure stress to guaranteed strength ratio (obtained using strength models presented herein) for the CFCC 1 \( \times \) 7 stirrups is 10.3 percent higher for anchorage type A, and 8.3 percent lower for anchorage type B in comparison to those obtained using Morphy et al. (1997) equations. However, as mentioned earlier, reduction in uni-axial stirrup strength due to bend effect is almost the same (40% reduction) as reported by Morphy et al. (1997).
Figures 4.4 and 4.5 show relationship between the ratio of failure stress to guaranteed strength and the ratio of embedded length to effective diameter for 10.5 mm (0.41 in.) diameter CFTC 1 x 7 stirrups and 9.5 mm (0.374 in.) diameter C-bar (MIC) stirrups, respectively. The relationships between the uni-axial stirrup strength and the embedded length for 10.5 mm (0.41 in.) diameter CFTC 1 x 7 stirrup and 9.5 mm (0.374 in.) diameter C-bar (MIC) stirrups with bend radius of 38 mm (1.5 in.) are given below:

10.5 mm (0.41 in.) diameter CFTC 1 x 7 stirrups with tail length 11d₀

**Anchor type A**

\[
\frac{f_u}{f_{tg}} = 0.35 \quad \text{for } \frac{l_d}{d_e} < 5.8 \quad (4.5a)
\]

\[
= 0.0251 \left( \frac{l_d}{d_e} \right) + 0.26 \quad \text{for } 5.8 \leq \frac{l_d}{d_e} \leq 29.5 \quad (4.5b)
\]

\[
= 1.0 \quad \text{for } \frac{l_d}{d_e} > 29.5 \quad (4.5c)
\]

**Anchor type B**

\[
\frac{f_u}{f_{tg}} = 0.35 \quad \text{for } \frac{l_d}{d_e} < 5.8 \quad (4.6a)
\]

\[
= 0.0394 \left( \frac{l_d}{d_e} \right) + 0.16 \quad \text{for } 5.8 \leq \frac{l_d}{d_e} \leq 21.3 \quad (4.6b)
\]

\[
= 1.0 \quad \text{for } \frac{l_d}{d_e} > 21.3 \quad (4.6c)
\]

9.5 mm (0.374 in.) diameter C-bar stirrups with tail length 12 d₀

**Anchor type A**

\[
\frac{f_u}{f_{tg}} = 0.24 \quad \text{for } \frac{l_d}{d_e} < 5.0 \quad (4.7a)
\]
\[
= 0.0171 \left( \frac{l_d}{d_e} \right) + 0.20 \quad \text{for } 5.0 \leq \frac{l_d}{d_e} \leq 15.0 \quad (4.7b)
\]

\[
= 1.0 \quad \text{for } \frac{l_d}{d_e} > 15.0 \quad (4.7c)
\]

**Anchor type B**

\[
\frac{f_u}{f_{tg}} = 0.32 \quad \text{for } \frac{l_d}{d_e} < 5.0 \quad (4.8a)
\]

\[
= 0.0164 \left( \frac{l_d}{d_e} \right) + 0.28 \quad \text{for } 5.0 \leq \frac{l_d}{d_e} \leq 44 \quad (4.8b)
\]

\[
= 1.0 \quad \text{for } \frac{l_d}{d_e} > 44 \quad (4.8c)
\]

Figures 4.6a to d show typical failure modes for 10.5 mm (0.41 in.) diameter CFCC $1 \times 7$ stirrup together with their concrete block anchorage systems, while Figures 4.7 a to d show typical failure modes for 10.5 mm (0.41 in.) diameter CFTC $1 \times 7$ stirrups with their concrete anchorage system. The slippage of stirrups in the concrete blocks (Figures 4.6a and 4.7a) is due to weak bond between the concrete and stirrups. The splitting of concrete in the plane of CFCC $1 \times 7$ stirrup (Figure 4.6b) occurred without rupture of stirrup.

Also, it is noted that the splitting of concrete occurred in the end concrete block, which had no debonding tube on the stirrups. Unlike 10.5 mm (0.41 in.) diameter CFCC $1 \times 7$ stirrups, most of the 10.5 mm (0.41 in.) diameter CFTC $1 \times 7$ stirrups failed due to rupture of the strands between the concrete blocks or at the end of debonding tube (Figure 4.7d). To avoid the undesirable failure mode such as splitting of concrete block without rupture of stirrup strands, concrete block containing stirrup end with no debonding must be reinforced using mild steel or FRP reinforcements (see Figure 3.6).
4.3 Evaluation of Stirrup Strength from Beam Tests

To examine the behavior of CFRP stirrups as shear reinforcements in concrete beams reinforced and prestressed using CFRP tendons, results of the prestressed concrete box-beams tested under shear load were utilized. A total of six 4,572 mm (15 ft) span and 305 mm (12 in.) deep box-beams were tested under shear load. Figure 4.8 shows the relationship between the ratio of effective stirrup stress to the guaranteed strength and the ratio of stirrup spacing to the effective depth for all tested box-beams reinforced with CFCC and C-bar (MIC) stirrups. As expected, stirrup provided at larger center-to-center spacing is stressed to greater stress value (Figure 4.8). The effective stress developed in stirrups was computed using equations (ACI 318, 2002), which are based on ultimate shear load, $V_u$ and shear strength contribution of concrete as measured by shear strength of the control beam. Based on the experimental results, the following equation is recommended for evaluating the effective stress in CFRP stirrups, which can be used further in the determination of shear strength contribution of stirrups in conjunction with uni-axial stirrup strength, $f_u$.

$$\frac{f_{fv}}{f_{tg}} = 0.3 \left( \frac{s}{d} \right) + 0.05 \leq 1.0 \quad \text{for } f_{fv} \leq f_u$$

(4.9)

where $f_{fv}$ is effective stress in stirrups at the beam failure; $f_{tg}$ is the guaranteed strength of stirrup bar, $d$ is the effective depth of box-beam, and $s$ is the center-to-center spacing of stirrups.

4.4 Bond Characteristics

In this section, bond characteristics of C-bars/tendon such as DCI, MIC, and Leadline (MCC) bars are presented along with those of steel. The bond capacity of the bar is based on the maximum load carrying capacity of T-beam. Two types of failure modes were encountered during the T-beam bond tests, i.e., bond failure and concrete compression failure. Here, bond failure is defined as the initiation of slippage of the free end of the bar, while compression failure is defined as the crushing of concrete at the top.
flange of the beam. The calculation of the bond stress was made using bending moment method and strain method as described in Appendix A.

### 4.4.1 Bond Characteristics of CFRP Bars

Figure 4.9 shows the typical load versus free-end slip relationships for DCI CFRP bar. The bond failure loads of nine T-beams corresponding to slips at ends A and B varied from 76 to 104 kN (17 to 23.3 kips). It should be noted that the bond length ($l_b$) is measured from the free end of the beam to the point where major crack intersected the reinforcing bar. The bond lengths of DCI bars varied from 222 to 457 mm (8.75 to 18 in.). It is observed from Figure 4.9 that the load carrying capacity of test T-beams does not increase significantly after the initiation of slippage of bars/tendons. The calculated bond strength of DCI bars varied from 6.6 to 9.5 MPa (0.95 to 1.37 ksi). The calculated bond strengths of DCI bars are lower than the maximum bond strength [11.5 MPa (1.67 ksi), Karlson, 1997] observed for C-bar specimens from the pull-out test. However, these values are about 3.1 to 4.4 times that of the average value observed for C-bar specimens with bond length equal to 2.5 times the diameter of bar (Nanni et al., 1997). Two bond strength results for the DCI bar were found to be inconsistent with the other bond test results and were discarded. The average value of the bond strength of DCI bars was 7.9 MPa (1.15 ksi), while that of C-bars (Nanni et al., 1997), the bond strength was 2.13 MPa (0.31 ksi). The corresponding load versus slip relationships for C-bar (MIC) is shown in Figure 4.10. The failure loads of the six tested T-beams with C-bars (MIC) varied from 76.5 to 94.3 kN (17.2 to 21.2 kips), while the bond length varied from 299 to 406 mm (11.75 to 16 in.). The corresponding bond strength of MIC C-bars varied from 6.5 to 8.3 MPa (0.94 to 1.21 ksi). The average value of the bond strengths of MIC C-bars was 6.895 MPa (1.0 ksi), which is about 3.2 times of that observed for C-bars (Nanni et al., 1997) of bond length of 2.5 times the bar diameter.

Figure 4.11 shows the typical load versus slip relationship for a Leadline (MCC) tendon. Based on the failure of ten tested T-beams with different embedment lengths, it was observed that the failure load varied from 71.2 to 103 kN (16 to 23.1 kips). Bond lengths of Leadline tendons varied from 238 to 540 mm (9.38 to 21.25 in.). The
corresponding bond strength varied from 5.1 to 8.2 MPa (0.74 to 1.19 ksi). One of the test results of Leadline tendon was found to be inconsistent with other bond strength results of Leadline tendons, i.e., bond strength of 10.3 MPa (1.5 ksi) was discarded. The average value of bond strength of Leadline tendon was 6.3 MPa (0.91 ksi), which is about 2.96 times that observed for C-bars (Nanni et al., 1997) of bond length equal to 2.5 times diameter of bar.

4.4.2 Bond Characteristics of Steel Bars

In addition to the bond characteristics of three different CFRP bars, bond characteristics of steel bars were also evaluated. The bond characteristics of steel bars provided the basis for comparison of bond characteristics of CFRP bars. For the six tested T-beams with steel bars, failure load varied from 22.3 to 36.5 kN (5 to 8.2 kips). The embedment of steel bars were smaller than those of CFRP bars. It was observed that the beam having embedment length of 50.8 mm (2 in.) failed in compression, while the other beams experienced bond failure. The typical load versus slip relationships for steel bar are shown in Figure 4.12. Bond length was equal to the full embedment length in all the beams and the calculated bond strength varied from 12.3 to 15.7 MPa (1.78 to 2.28 ksi). The average value of the bond strength of steel bar was 14.5 MPa (2.1 ksi), which is about 9 times that of 420 MPa steel bar of bond length 2.5 times bar diameter [13 mm (0.51 in.)] (Nanni et al., 1997). This high difference in bond strengths of steel is due to different test methods employed to predict the bond strengths and the bar diameters. Nanni et al. (1997) used pull-out test to predict the bond strength of 0.51 in. (13 mm) steel, whereas in the present investigation, T-beam test was used to measure corresponding bond strength of 0.374 in. (9.5 mm) diameter steel bar having yield strength of 60 ksi (414 MPa).

Bond strengths, bond lengths, and failure modes are summarized in Table 4.3. In the table, TD, TM, TL, and TS refers to T-beams reinforced with DCI, MIC, Leadline, and steel bars, respectively, while the numbers, immediately following the letter represent the embedment lengths (in.) and beam designation number, respectively. The average bond strengths of different bars/tendons are presented in Table 4.4 for the concrete
Table 4.3 Test results of T-Beam bond specimens

<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Bar Material</th>
<th>Nominal Bar Diameter, d (in.)</th>
<th>Failure Load kN (kips)</th>
<th>Bond Length, l_b (mm) (in.)</th>
<th>Bond Strength, U MPa (ksi)</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>TD-30*1</td>
<td>DCI</td>
<td></td>
<td>103.7 (23.3)</td>
<td>235 (9.25)</td>
<td>9.38 (1.360)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-30-2</td>
<td></td>
<td></td>
<td>95.7 (21.5)</td>
<td>406.4 (16.00)</td>
<td>7.24 (1.050)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-30-3</td>
<td></td>
<td></td>
<td>97.5 (21.9)</td>
<td>241.3 (9.50)</td>
<td>11.23 (1.630)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-24-1</td>
<td></td>
<td></td>
<td>80.5 (18.1)</td>
<td>292.1 (11.50)</td>
<td>7.21 (1.045)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-24-2</td>
<td></td>
<td></td>
<td>93.5 (21.0)</td>
<td>374.7 (14.75)</td>
<td>6.55 (0.950)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-24-3</td>
<td></td>
<td></td>
<td>76.1 (17.1)</td>
<td>222.3 (8.75)</td>
<td>9.45 (1.370)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-18-1</td>
<td></td>
<td></td>
<td>75.7 (17.0)</td>
<td>279.4 (11.00)</td>
<td>8.00 (1.160)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-18-2</td>
<td></td>
<td></td>
<td>76.5 (17.2)</td>
<td>457.2 (18.00)</td>
<td>4.654 (0.675)</td>
<td>Bond</td>
</tr>
<tr>
<td>TD-18-3</td>
<td></td>
<td></td>
<td>---</td>
<td>----</td>
<td>---</td>
<td>Compression</td>
</tr>
<tr>
<td>TM-30-1</td>
<td>MIC</td>
<td></td>
<td>89.9 (20.2)</td>
<td>368.3 (14.50)</td>
<td>7.38 (1.070)</td>
<td>Bond</td>
</tr>
<tr>
<td>TM-30-2</td>
<td></td>
<td></td>
<td>89.4 (20.1)</td>
<td>406.4 (16.00)</td>
<td>6.27 (0.910)</td>
<td>Bond</td>
</tr>
<tr>
<td>TM-24-1</td>
<td></td>
<td></td>
<td>89 (20.0)</td>
<td>311.2 (12.25)</td>
<td>6.48 (0.940)</td>
<td>Bond</td>
</tr>
<tr>
<td>TM-24-2</td>
<td></td>
<td></td>
<td>94.3 (21.2)</td>
<td>298.5 (11.75)</td>
<td>8.34 (1.210)</td>
<td>Bond</td>
</tr>
<tr>
<td>TM-18-1</td>
<td></td>
<td></td>
<td>90.8 (20.4)</td>
<td>323.9 (12.75)</td>
<td>7.31 (1.060)</td>
<td>Bond</td>
</tr>
<tr>
<td>TM-18-2</td>
<td></td>
<td></td>
<td>76.5 (17.2)</td>
<td>304.8 (12.00)</td>
<td>6.69 (0.970)</td>
<td>Bond</td>
</tr>
</tbody>
</table>

* Represents the embedment of 30 in. for Beam TD-30-1. All beams were fabricated with the concrete having strength of 7 ksi.
<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Bar Material</th>
<th>Nominal Bar Diameter, d (mm (in.))</th>
<th>Failure Load kN (kips)</th>
<th>Bond Length, l, mm (in.)</th>
<th>Bond Strength, U MPa (ksi)</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>TL-30-1</td>
<td>Leadline</td>
<td>93.5 (21.0)</td>
<td>406.4 (16.00)</td>
<td>7.05 (1.022)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-30-2</td>
<td>Leadline</td>
<td>71.2 (16.0)</td>
<td>238.3 (9.38)</td>
<td>5.86 (0.850)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-30-3</td>
<td>Leadline</td>
<td>93.5 (21.0)</td>
<td>539.8 (21.25)</td>
<td>5.51 (0.799)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-24-1</td>
<td>Leadline</td>
<td>94.0 (21.1)</td>
<td>374.7 (14.75)</td>
<td>7.14 (1.035)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-24-2</td>
<td>Leadline</td>
<td>84.6 (19.0)</td>
<td>336.6 (13.25)</td>
<td>5.93 (0.860)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-24-3</td>
<td>Leadline</td>
<td>102.8 (23.1)</td>
<td>387.4 (15.25)</td>
<td>10.34 (1.500)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-18-1</td>
<td>Leadline</td>
<td>85.0 (19.1)</td>
<td>406.4 (16.00)</td>
<td>5.24 (0.760)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-18-2</td>
<td>Leadline</td>
<td>89.0 (20.0)</td>
<td>412.8 (16.25)</td>
<td>5.1 (0.740)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-18-3</td>
<td>Leadline</td>
<td>91.2 (20.5)</td>
<td>241.3 (9.50)</td>
<td>6.55 (0.950)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TL-18-4</td>
<td>Leadline</td>
<td>97.9 (22.0)</td>
<td>285.8 (11.25)</td>
<td>8.21 (1.190)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TS-2-1</td>
<td>Steel</td>
<td>26.3 (5.9)</td>
<td>8.9 (2.0)</td>
<td>12.34 (1.789)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TS-2-2</td>
<td>Steel</td>
<td>23.0 (5.17)</td>
<td>38.0 (1.5)</td>
<td>15.77 (2.287)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TS-1.5-1</td>
<td>Steel</td>
<td>22.5 (5.05)</td>
<td>38.0 (1.5)</td>
<td>15.18 (2.201)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TS-1.5-2</td>
<td>Steel</td>
<td>34.8 (7.82)</td>
<td>25.4 (1.0)</td>
<td>49.0 (7.113)</td>
<td>Bond</td>
<td></td>
</tr>
<tr>
<td>TS-1-2</td>
<td>Steel</td>
<td>36.4 (8.19)</td>
<td>25.4 (1.0)</td>
<td>35.3 (5.120)</td>
<td>Bond</td>
<td></td>
</tr>
</tbody>
</table>
Table 4.4  Average bond strengths for bars and tendons

<table>
<thead>
<tr>
<th>Bar Material</th>
<th>Nominal Bar Diameter, d mm (in.)</th>
<th>( f_c' ) MPa (ksi)</th>
<th>Average Bond Strength MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCI</td>
<td>9.5 (0.374)</td>
<td></td>
<td>7.930 (1.15)</td>
</tr>
<tr>
<td>MIC</td>
<td>9.5 (0.374)</td>
<td>48 (7.0)</td>
<td>6.900 (1.00)</td>
</tr>
<tr>
<td>Leadline</td>
<td>10 (0.394)</td>
<td></td>
<td>6.270 (0.91)</td>
</tr>
<tr>
<td>Steel</td>
<td>9.5 (0.374)</td>
<td></td>
<td>14.480 (2.10)</td>
</tr>
</tbody>
</table>
having compressive strength of 48 MPa (7 ksi). In the table, bond failure refers to the failure of beam due to slippage of tendon, whereas compression failure refers to the failure of T-beam due to crushing of concrete. Note that three typical failure modes were observed in the T-beam bond test. These failure modes are flexural failure (Figure 4.13), shear/flexural failure (Figure 4.14), and shear failure (Figure 4.15). Flexural flexure occurred due to propagation of flexure crack in the midspan region, whereas flexure/shear failure occurred in the shear zone near the support where the crack initiated as shear crack and then vertically propagated as flexure crack. The shear failure (Figure 4.15) of bond test T-beam occurred due to propagation of shear crack near the support.

4.5 Transfer Lengths

In this section, transfer lengths for all the nine box-beams (six tested for shear and three tested for flexure under ultimate load test) are presented. These transfer lengths are based on the saw-cut release of prestressing forces 7 days after concrete was poured. The strength of concrete after seven days was approximately 42.7 MPa (6.2 ksi). Transfer lengths were determined from the strain distribution along the length of the beam near the ends using 95% average maximum strain method for 43%, 57%, and 100% of release of prestressing forces. The 43% release of prestressing forces refers to the saw-cut of three pretensioning tendons; 57% release of prestressing forces refers to the saw-cut of four pretensioning tendons, and 100% release of prestressing forces refers to the saw-cut of all seven pretensioning tendons. The strain distribution and measured transfer lengths for beams M2, M3, T2, T3, S2, and N0 are shown in Figures 4.16 through 4.24, respectively. In the designations of box-beams, letters M, T, and S refer to the beams provided with MIC C-bar, Tokyo Rope (CFCC), and steel stirrups, whereas numbers 2 and 3 associated with each beam represent the stirrup spacings of d/2 and d/3, respectively. Here, d is the effective depth of the box-beam. Beam N0 refers to box-beam without shear stirrups in the shear span. Details of prestressing forces at transfer, measured transfer lengths, and the calculated transfer lengths (obtained using transfer length Eq. 4.1, Grace, 2000a) are presented in Table 4.5 for six box-beams tested for shear and three box-beams (DP1, DN2, and LP3) tested for flexure. It is observed that the amount of transverse reinforcement has little effect on the measured transfer length, since box-beams tested
had different number and type of stirrups at each end. It is also observed that the average values of the measured transfer lengths at dead and live ends are lower than that predicted using transfer length equation of Grace (2000a) by about 21 to 43%. Since each tested beam had an end 965 (long) × 305 (wide) × 305 (deep) mm [38 (long) × 12 (wide) × 12 in. (deep)] cross-beam, the measured transfer length was close to 12 in. (width of the cross-beam). Also, the presence of the transverse beam (cross-beam) at each end eliminated the possibility of developing any longitudinal cracks at release.

\[
L_t = \frac{f_{pi} d_b}{\alpha_i f_{ci}^{0.67}} 
\]  
(Grace, 2000a)  
(4.1)

where \(d_b\) is the nominal diameter of tendon/strand (mm), \(f_{pi}\) is prestress at transfer (MPa); \(f_{ci}\) is the strength of concrete at transfer (MPa); and \(\alpha_i\) is transfer length coefficient, where \(\alpha_i = 1.95\) for Leadline tendons and 2.12 for CFCC strands.

### 4.6 Shear Response of Box-Beams

In this section, responses of beams M2, M3, T2, T3, and S2 under applied shear load are discussed. Figure 4.25 shows the shear force versus stirrup strain relationships. It is observed that for a specific shear force, stirrups spaced at greater center-to-center distance experiences greater strain. The maximum strains developed in stirrups of beams M2, M3, T2, T3, and S2 are 0.004, 0.003, 0.0038, 0.0025, and 0.0019 (23.5%, 17.6%, 25%, 16.7%, and 95% of the corresponding ultimate strain of stirrups), respectively. Thus, it is obvious that the development of strain in stirrups in the beams depends on the spacing of stirrups. It is also observed that MIC and CFCC stirrups at spacing of half the effective depth of the beams had similar responses as that of steel stirrups.

Figures 4.26 through 4.28 show the typical distribution of shear cracks in box-beams M2, M3, and T3, respectively. It should be noted that the cracks under applied load first developed as flexural cracks, however with the increase in the shear force, shear
<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Average Prestressing Force, kN (kips)</th>
<th>Tendon Stress, MPa (ksi)</th>
<th>End Slip, mm (in.)</th>
<th>Transfer Length, mm (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Dead End</td>
<td>Live End</td>
</tr>
<tr>
<td>Shear-Beams</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M2</td>
<td>92.1 (20.7)</td>
<td>1298 (188)</td>
<td>3.05</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M3</td>
<td>92.1 (20.7)</td>
<td>1298 (188)</td>
<td>2.03</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T2</td>
<td>90.8 (20.4)</td>
<td>1279 (186)</td>
<td>4.57</td>
<td>2.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T3</td>
<td>91.7 (20.6)</td>
<td>1291 (187)</td>
<td>4.57</td>
<td>2.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>90.3 (20.3)</td>
<td>1272 (185)</td>
<td>3.05</td>
<td>3.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N0</td>
<td>90.3 (20.3)</td>
<td>1272 (185)</td>
<td>0.51</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural-Beams</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DP1</td>
<td>93.9 (21.1)</td>
<td>1323 (192)</td>
<td>0.76</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DN2</td>
<td>93.5 (21.0)</td>
<td>1316 (191)</td>
<td>2.03</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LP3</td>
<td>94.8 (21.3)</td>
<td>1204 (175)</td>
<td>0.76</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Calculated transfer lengths, Grace (2000a).
cracks developed in the shear critical zone. The shear-cracking load is defined as the shear force at which first change in the stirrup strain occurred.

### 4.6.1 Shear Crack Widths

The shear cracks started as vertical cracks at the loading points, but with increase in the load, inclined cracks started appearing. Figure 4.29 shows the shear force versus crack width relationships for all the six box-beams tested under shear load. The crack width was calculated using Eq. 4.2 (Shehata, 1999) as given below.

\[
\sum w = \left( \sqrt{2} \Delta_D - \Delta_V - 0.5 l_g \varepsilon_{ct} \right) \sin \theta + \left( \sqrt{2} \Delta_D - 0.5 l_g \varepsilon_{ct} \right) \cos \theta
\]

where, \( \sum w \): the summation of all shear crack widths, \( \Delta_D \): the diagonal displacement measured using DEMEC gages, \( \Delta_V \): the vertical displacement measured using DEMEC gages, \( \varepsilon_{ct} \): maximum tensile strain of concrete (0.0001), \( l_g \): the gage length of DEMEC gages, and \( \theta \): the angle of shear crack with the horizontal axis.

Shear cracks started appearing between 178 kN (40 kips) and 267 kN (60 kips) for all the tested beams after the cracks initiated as vertical cracks. The initial shear cracks were not visible to the naked eye. However, at higher loads, the inclined shear cracks grew to the visible state. Failure shear cracks for all the tested beams were inclined at angle between 45° and 47°. It can be observed from Figure 4.29 that the crack widths for all beams except beam N0 showed similar pattern. Box-beams T2 and T3 had better crack control compared to other beams. This is attributed to the better bond properties of Tokyo Rope stirrups. In the box-beam without shear reinforcement (N0), the quick growth of cracks resulted in premature failure of the beam. Obviously, this observation confirms the importance of stirrups with regard to improving the shear capacity of beams. The observed crack behavior of all other beams were similar to that reinforced with steel stirrups. This observation is consistent with Shehata’s (1999) observation. Thus, it is noted that the CFRP stirrups are as effective in the shear crack control as steel stirrups.
4.6.2 Ultimate Load Response of Box-Beams

Figures 4.30-4.35 show the load versus deflection relationships for box-beams M2, M3, T2, T3, S2, and N0, respectively, while Figure 4.36 shows the comparison of the load-deflection responses of all six tested box-beams. In Figures 4.30 through 4.35, different load tests before the ultimate load test, represent the loading and unloading cycle corresponding to a particular load test. For example, 50 kips load test represents that the beam was loaded to 50 kips and then unloaded to zero load level. Finally, after all loading and unloading cycles, beams were subjected to ultimate load test to predict ultimate load response such as load carrying capacity of beams and deflection corresponding to the ultimate load. These loading and unloading cycles are required to evaluate the energy ratio as a measure of ductility of beam. The measurement of energy ratio is explained later in the section entitled “ductility.” It should be noted that the deflection of all the beams increased significantly after development of cracks. As observed from Figure 4.36, the deformation of beams reinforced with CFRP stirrups are comparable to that of the beam reinforced with steel stirrups. This observation is consistent with Shehata’s observation that the variation of stirrup material and spacing did not significantly affect the load-deflection characteristics. The deflection corresponding to the ultimate failure of beams reinforced with CFRP stirrups spaced at d/2 is more than the corresponding deflection of beam (S2) reinforced with steel stirrups. It is also observed from Figure 4.36 that beam T3 had the best load deflection response in comparison to that of other beams. This is attributed to the better crack control characteristics of beam T3 in comparison to the other beams.

It should also be noted that all six box-beams, as expected, failed in shear. The ultimate shear load capacities of beams M2, M3, T2, T3, S2, N0 were 257, 267, 226, 291, 223, and 177 kN (57.8, 60.0, 50.9, 65.5, 50.1, and 40 kips), respectively. The shear failure of all the beams was caused due to widening of shear cracks followed by rupture of the pretensioning tendons due to the dowel action. As mentioned earlier, cracks were inclined at an angle between 45° to 47° from the longitudinal axis of beams. Also, it was observed that the stirrups of the beams did not rupture, and hence the full strength of
stirrups could not be utilized. The typical shear failure of box-beams M2, M3, T2, T3, S2, and N0 is shown in Figures 4.37 through 4.42, respectively.

4.6.3 Forces in Post-tensioning Tendons

Figures 4.43-4.47 show the variation of forces in the post-tensioning tendons with the increase in applied loading. It is observed that there is no significant effect of applied load on the post-tensioning forces up to shear cracking loads [178-223 kN (40 -50 kips)] of the beams. After the initial cracks, beam deformed considerably, which resulted in about 20% increase in forces in the post-tensioning unbonded tendons. It must be noted that the unbonded post-tensioning tendons did not rupture at the instant of ultimate failure of box-beams. Thus, the post-tensioning tendons effectively contributed to the strengthening of box-beams.

4.6.4 Ductility of Box-Beams

Since the ductility of CFRP prestressed and/or reinforced concrete structures has always been an issue of concern due to low elastic modulus and brittle failure of CFRP bars/tendons, experimental approach was used to evaluate the ductility of beams. As mentioned earlier, the box-beams were subjected to loading and unloading cycles to determine the inelastic energy absorbed in each tested beams before their ultimate failure. The inelastic energy was used to compute the energy ratio as a measure of ductility. Energy ratio is defined as the ratio of inelastic energy absorbed in the box-beam to the total energy (sum of elastic and inelastic energy). The evaluation of energy ratios for beams M2, M3, T2, T3, S2, and N0 are shown in Figures 4.48-4.53, respectively. Elastic energy was evaluated by calculating the area enclosed by a curve parallel to the last unloading cycle and passing through the ultimate load of the beam. It is observed that beam S2 has the highest energy ratio while beam N0 has the lowest energy ratio. The elastic and inelastic energy, and energy ratios for beams M2, M3, T2, T3, and S2, and N0 are presented in Figures 4.48 through 4.53, respectively. Details of ultimate shear strengths, average stirrup strains, shear cracking loads, and energy ratios for all the box-beams are presented in Table 4.6. It should be noted that beams reinforced with MIC C-bar stirrups have higher ductility than those reinforced with CFCC stirrups. Note that the
energy ratio of CFRP reinforced beam remains almost unaffected by changing the stirrup spacing from d/2 to d/3. It should also be noted that for box-beam T2, the loading and unloading near the failure load was not possible. Therefore, an assumed loading curve parallel to the unloading curve of 80 kips load test (Figure 4.50) was used. Thus, the energy ratio for beam T2 is based on this assumption.

4.7 Flexural Response of Box-Beams

In this section, the flexural response of three box-beams DP1, DN2, and LP3 is discussed in detail. As mentioned earlier, beam DP1 refers to the flexural box-beam #1 prestressed using bonded and unbonded DCI tendons; beam DN2 refers to the flexural box-beam #2 prestressed using bonded DCI pretensioning tendons without post-tensioning forces in unbonded DCI tendons; and beam LP3 refers to the flexural box-beam #3 prestressed using bonded pretensioning and unbonded post-tensioning Leadline (MCC) tendons. In order to predict the overall flexural response, it was necessary to have the knowledge of cracking loads of the three-box-beams. Based on the visual inspection of the initially developed cracks, the cracking loads of Beams DP1, DN2, and LP3 were detected to be 169, 89, and 187 kN (38, 20, and 42 kips), respectively. The cracking loads could be clearly identified from the load deflection curves of the beams to be discussed later. It should be noted that initial cracks led to the developments of the fully-grown flexural cracks, which propagated rapidly to cause the ultimate failure of beam. It was observed that the crack pattern was similar and almost identical on both sides of the beam indicating a uniform distribution of load across the beam. A typical flexural crack distribution pattern in the midspan zone is shown in Figure 4.54 for box-beam LP3.

4.7.1 Decompression Load

In order to determine the actual effective prestress in the pretensioning tendons, decompression load was predicted using load-strain relationships for the box-beam obtained for the sections where initial crack developed. The strain readings were obtained from the strain gages installed on the bottom flange of the beam (see Figure 3.29
Table 4.6  Test results for box-beams tested for shear strengths

<table>
<thead>
<tr>
<th>Beam</th>
<th>Stirrups</th>
<th>Spacing</th>
<th>Shear Cracking Force, kN (kips)</th>
<th>Angle of Major Crack deg. (θ)</th>
<th>Ultimate Shear, kN (kips)</th>
<th>Average Stirrup Strain at Failure %</th>
<th>Energy Ratio %</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2</td>
<td>MIC</td>
<td>d/2</td>
<td>111 (25)</td>
<td>45</td>
<td>257 (57.8)</td>
<td>0.300</td>
<td>24</td>
</tr>
<tr>
<td>M3</td>
<td>MIC</td>
<td>d/3</td>
<td>156 (35)</td>
<td>47</td>
<td>267 (60.0)</td>
<td>0.400</td>
<td>23</td>
</tr>
<tr>
<td>T2</td>
<td>CFCC</td>
<td>d/2</td>
<td>111 (25)</td>
<td>46</td>
<td>226 (50.9)</td>
<td>0.250</td>
<td>17</td>
</tr>
<tr>
<td>T3</td>
<td>CFCC</td>
<td>d/3</td>
<td>156 (35)</td>
<td>45</td>
<td>291 (65.5)</td>
<td>0.379</td>
<td>17</td>
</tr>
<tr>
<td>S2</td>
<td>Steel</td>
<td>d/2</td>
<td>133 (30)</td>
<td>45</td>
<td>223 (50.1)</td>
<td>0.190</td>
<td>25</td>
</tr>
<tr>
<td>N0</td>
<td>N/A</td>
<td>N/A</td>
<td>80 (18)</td>
<td>46</td>
<td>177 (40)</td>
<td>N/A</td>
<td>15</td>
</tr>
</tbody>
</table>

# d is the effective depth of box-beam and equals to 254 mm (10 in.); N/A refers to not-applicable results.
Table 4.7  Details of prestressing forces and failure loads of box-beams tested for flexure

<table>
<thead>
<tr>
<th>Beam Notation</th>
<th>Number and type of tendons used in prestressing</th>
<th>Total Prestressing Force, kN (kips)</th>
<th>(E_f/E_s)*</th>
<th>f_{fu}**</th>
<th>Ultimate load, kN (kips)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pretensioning</td>
<td>Post-tensioning</td>
<td>Pretensioning</td>
<td>Post-tensioning</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DP1</td>
<td>7 DCI tendons</td>
<td>6 DCI tendons</td>
<td>1206 (271)</td>
<td>0.655</td>
<td>1931 (280)</td>
<td>383 (86)</td>
</tr>
<tr>
<td>DN2</td>
<td>7 DCI tendons</td>
<td>6 DCI tendons</td>
<td>654 (147)</td>
<td>0.655</td>
<td>1931 (280)</td>
<td>307 (69)</td>
</tr>
<tr>
<td>LP3</td>
<td>7 Leadline (MCC) tendons</td>
<td>6 Leadline (MCC) tendons</td>
<td>1202 (270)</td>
<td>0.735</td>
<td>2,861 (415)</td>
<td>441 (99)</td>
</tr>
</tbody>
</table>

* E_f and E_s refer to Young’s modulus of elasticity of CFRP tendons and steel, respectively; ** f_{fu} refers to the specified strength of CFRP tendons. Note: The 28-day compressive strength of concrete, f_c is equal to 48 MPa (7 ksi).
The gages were installed adjacent to first developed crack while holding the load constant and were connected to the data acquisition system. Loading of the beam was resumed to gather concrete strain readings. As indicated by the data presented in Figure 4.55, the load versus strain plot exhibited a gradual transition and then became nearly constant. The applied load, where the strain ceases to increase further is judged to be an indication that all the precompression in the beam has been overcome. Thus, decompression load was predicted as the load at which no further increase in the strain occurred. As shown in the Figure 4.55, predicted decompression loads for the beams DP1, DN2, and LP3 are 140, 84, and 142 kN (31.5, 18.8, and 32 kips), respectively. The lower value of decompression load for box-beam DN2 is due to the lack of post-tensioning forces in the unbonded post-tensioning tendons. The total level of prestressing forces in beam DN2 was 54% of that in the case of beams DP1 and LP3. The prestress losses were computed using back calculation method (see Appendix A), wherein decompression loads were used to calculate actual stress at the bottom of the beam due to effective prestress. The computed prestress losses in pretensioning forces were about 13.6%, 8.3%, and 10.5% for beams DP1, DN2, and LP3, respectively. Details of prestressing levels, failure loads and failure modes of all the three beams are summarized in Table 4.7.

4.7.2 Concrete Strains due to Prestressing

Figure 4.56 shows the strain distributions in the midspan cross-sections of all the nine box-beams (six tested for shear response and three tested for flexural response). Due to same level and method of prestressing, strain distributions of all the beams are similar. It is also shown that the applied prestressing resulted into sufficient compressive concrete strain in the tensile zone to counteract the tensile stresses caused due to service loads.

4.7.3 Load-Deflection Response

The load versus deflection relationships for beams DP1, DN2, and LP3 are shown in Figure 4.57. It is shown that beam DN2, without post-tensioning forces, underwent substantial deformation in comparison to that of other beams. It is also shown that the load-deflection relationships are bilinear before the ultimate failure. As expected, the
flexural stiffness of box-beams reduced after their corresponding cracking load. The end of the initial slope of the load-deflection relationship marks the cracking load for a beam. The midspan deflections corresponding to the ultimate failure of beams LP3, DP1, and DN2 are 58.7, 65.0, and 73.0 mm (2.31, 2.56, and 2.875 in.), respectively. In order to examine the effect of loading-unloading cycles on the load-deflection response, the load versus deflection responses of beam DN2 [for various loading ranges, i.e., 178, 312 kN (40, 70 kips), and failure load] are shown in Figure 4.58. It is observed that the beam underwent a permanent deformation after loading-unloading cycle applied beyond the cracking load of box-beams. However, the loading-unloading cycles have no significant effect on the load-deflection characteristics of the beams.

4.7.4 Post-tensioning Forces

Figure 4.59 shows the variation of forces of unbonded post-tensioning tendons of box-beams DP1, DN2, and LP3. It is observed that there is no significant change in the forces of unbonded post-tensioning tendons up to cracking load; however, post-tensioning forces increased by about 15 and 20 percent in beams DP1 and LP3, respectively. In case of beam DN2, post-tensioning forces increased by about 27 kN (6 kips). As expected, initial values of post-tensioning forces in DP1 and LP3 tendons differed from that of DN2 tendons by about 20 kips (initial applied post-tensioning force).

4.7.5 Ductility

As in the case of box-beams tested for shear, ductility was determined for the box-beams tested for flexural response by evaluating the elastic and inelastic energy associated with each beam. Ductility of each beam is expressed by the energy ratio, i.e., the ratio of inelastic energy absorbed to the total energy of the beam. Figures 4.60-4.62 show the load-deflection curves used to compute the energy ratios of the beams. The computed energy ratios were 32%, 50%, and 33% for beams DP1, DN2, and LP3, respectively. It should be noted that the load carrying capacity of beams prestressed using bonded pretensioning and unbonded post-tensioning tendons is higher than that of beams prestressed using pretensioning tendons only. However, the computed energy ratio is
higher for the beam without prestressing forces in the unbonded post-tensioning tendons in comparison to that had prestressing forces in the post-tensioning tendons. The higher level of ductility of the beams without prestressing in the unbonded post-tensioning tendon is due to early rupture of the bonded pretensioning tendons followed by delayed crushing of concrete giving additional inelastic energy to the box-beam. The additional inelastic energy was negligible for the beams DP1 and LP3, which failed immediately after rupture of the bonded pretensioning tendons. The initiation of failure of box-beams due to rupture of bonded pretensioning tendons is due to the fact that the beam cross-sections were under-reinforced. Overall, energy ratios of the box-beams were less than or equal to 50%, which implies that these box-beams could not be considered as ductile structures, which prevents the sudden collapse of structures. Designing over-reinforced sections, which can alleviate the premature failure of bonded tendons before crushing of concrete, could increase the ductility of the box-beams (Grace, 2000b).

4.7.6 Failure Modes

All three test box-beams failed in flexure. As expected, the beams prestressed using both the pretensioning and post-tensioning tendons had higher load carrying capacity in comparison to that of prestressed box-beams using pretensioning tendons only without prestressing forces in the unbonded post-tensioning tendons. The failure loads of beams DP1, DN2, and LP3 were 383, 307, and 441 kN (86, 69, and 99 kips), respectively. However, from the ultimate load test results, it is obvious that the Leadline (MCC) tendons are more effective than DCI tendons with regard to improving the load carrying capacity of beams. This is attributed to the higher tensile strength of the Leadline tendons compared to the DCI tendons. The typical flexural failures of beams DP1, DN2, and LP3 are shown in Figures 4.63, 4.64, and 4.65, respectively. It should be noted that unlike box-beams tested in shear, the box-beams tested for flexure did not collapse suddenly to the ground, but remained suspended between supports (Figures 4.63-4.65). In fact, the presence of the external post-tensioning tendons helped in holding the beam from sudden collapse even after the rupture of pretensioning tendons. The failure of the pretensioning tendons was abrupt and the post-tensioning tendons were intact even after the ultimate failure of the beam.
4.7.7 Strain in the Prestressing Tendons

The relationship between the strain in the prestressing tendons and the applied load is shown in Figure 4.66. As shown in the figure, the variation of strain in tendons was not significant at beginning of the loading. However, at the decompression load levels, i.e., at the load of 140 kN (31.5 kips) for beam DP1 and 142 kN (32 kips) for beam LP3, the slope of the load-strain curves became less steep and strain values increased significantly at the ultimate failure of the beams. The strains in typical pretensioning tendons of beams DP1 and LP3 at the corresponding decompression loads were 0.0075 and 0.01, respectively. The strains in pretensioning tendons at cracking loads [169 kN (38 kips) for beam DP1 and 187 kN (42 kips) for beam LP3] of beams DP1 and LP3 were 0.008 and 0.011, respectively. It is also shown that DCI tendons of beam DP1 failed at strain level of 0.014, while that of beam LP3 failed at the strain level of 0.019.
Figure 4.1 Typical stress-strain relationships for 7.5 mm diameter CFCC $1 \times 7$ and CFTC $1 \times 7$ stirrups with tail length of $11d_b$, embedded length of $15d_b$ and anchor type A
Figure 4.2  Effect of embedded length, $l_d$, on strength of 7.5 mm diameter CFCC 1 x 7 stirrups with tail length of $11d_b$. 

\[ \frac{f_u}{f_y} = 0.0331 \frac{l_d}{d_e} + 0.46 \]

\[ \frac{f_u}{f_y} = 0.0297 \frac{l_d}{d_e} + 0.44 \]
Figure 4.3  Effect of embedded length, $l_d$, on strength of 7.5 mm diameter CFTC 1 x 7 stirrups with tail length of $11d_b$.
Figure 4.4 Effect of embedded length, $l_d$, on strength of 10.5 mm diameter CFTC $1 \times 7$ stirrups with tail length of $11d_b$.
Figure 4.5 Effect of embedded length, $l_d$, on strength of 9.5 mm diameter C-Bar stirrups with tail length of 12$d_b$

\[ \frac{f_u}{f_y} = 0.0164 \frac{l_d}{d_e} + 0.16 \]

\[ \frac{f_u}{f_y} = 0.0251 \frac{l_d}{d_e} + 0.26 \]
Figure 4.6  Typical failure modes of 10.5 mm diameter CFCC 1 × 7 stirrups and concrete blocks

(a) Slippage failure

(b) Splitting of concrete block

(c) Rupture of stirrups

(d) Failure of concrete cube

Figure 4.6 Typical failure modes of 10.5 mm diameter CFCC 1 × 7 stirrups and concrete blocks
Figure 4.7 Typical failure modes of 10.5 mm diameter CFTC $1 \times 7$ stirrups

(a) Slippage failure

(b) Splitting of concrete blocks

(c) Rupture of stirrups

(d) Failure of stirrup at the end of debonding tube
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Figure 4.47 Load versus post-tensioning force for tendons of box-beam S2
Energy ratio = \frac{76}{321} = 24\% 

Total energy = 321 kip-in.

Inelastic energy = 76 kip-in.

Elastic energy = 245 kip-in.

Figure 4.48  Energy ratio for box-beam M2
Energy ratio = \( \frac{78}{338} = 23\% \)

Total energy = 338 kip-in.

Inelastic energy = 78 kip-in.

Elastic energy = 260 kip-in.

Figure 4.49  Energy ratio for box-beam M3
Energy ratio = 50/297 = 17%

Total energy = 297 kip-in.

In elastic energy = 50 kip-in.

Elastic energy = 247 kip-in.

Figure 4.50 Energy ratio for box-beam T2
Energy ratio = \( \frac{59}{351} = 17\% \)

Total energy = 351 kip-in.

Inelastic energy = 59 kip-in.

Elastic energy = 292 kip-in.

Figure 4.51 Energy ratio for box-beam T3
Energy ratio = $\frac{56}{226} = 25\%$

Total energy = 226 kip-in.

Inelastic energy = 56 kip-in.

Elastic energy = 170 kip-in.

Figure 4.52 Energy ratio for box-beam S2
Figure 4.53  Energy ratio for box-beam N0

Energy ratio = 19/131 = 15%

Total energy = 131 kip-in.

Elastic energy = 112 kip-in.

Inelastic energy = 19 kip-in.
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After 57% release of pretensioning force | After 100% release of pretensioning force | After applying 100% of post-tensioning force

Strain distribution at midspan for beam T2

After 57% release of pretensioning force | After 100% release of pretensioning force | After applying 100% of post-tensioning force

Strain distribution at midspan for beam T3

Figure 4.56 (cont’d) Typical concrete strain (µε) distribution for box- beams due to release of pretensioning forces and application of post-tensioning forces
After release of 57% of pretensioning force  
After release of 100% of pretensioning force  
After applying 100% of post-tensioning force

Strain distribution at midspan for beam S2

After release of 57% of pretensioning force  
After release of 100% of pretensioning force  
After applying 100% of post-tensioning force

Strain distribution at midspan for beam N0

Figure 4.56 (cont’d) Typical concrete strain ($\mu$ε) distribution for box- beams due to release of pretensioning forces and application of post-tensioning forces
After release of 57% of pretensioning force

After release of 100% of pretensioning force

After applying 100% of post-tensioning force

Strain distribution at midspan for DP1

After release of 57% of pretensioning force

After release of 100% of pretensioning force

No post-tensioning force was applied

Strain distribution at midspan for DN2

Figure 4.56 (cont’d) Typical concrete strain ($\mu_e$) distribution for box- beams due to release of pretensioning forces and application of post-tensioning forces
After release of 57% of pretensioning force

After release of 100% of pretensioning force

After applying 100% of post-tensioning force

Strain distribution at midspan for beam LP3

Figure 4.56 (cont’d) Typical concrete strain ($\mu\varepsilon$) distribution for box- beams due to release of pretensioning forces and application of post-tensioning forces
Figure 4.57  Load versus midspan deflection of box-beams

Cracking load of beam DP1 = 38 kips
Cracking load of beam DN2 = 20 kips
Cracking load of beam LP3 = 42 kips
Figure 4.58  Load versus midspan deflection of box-beam DN2
Figure 4.59  Load versus post-tensioning force in unbonded tendons of box-beams
Energy ratio = 60/187 = 32%
Total energy = 187 kip-in.
Inelastic Energy = 60 kip-in.
Elastic Energy = 127 kip-in.

Figure 4.60 Energy curves for box-beam DP1
Energy ratio = 89/179 = 50%

Total energy = 179 kip-in.

Elastic energy = 90 kip-in.

Inelastic energy = 43 kip-in.

Additional Inelastic energy

Load (kips)

Deflection (in.)

Figure 4.61  Energy curves for box-beam DN2
Energy ratio = 61/186 = 33%
Total energy = 186 kip-in.
Inelastic energy = 61 kip-in.
Elastic energy = 125 kip-in.

Figure 4.62 Energy curves for box-beam LP3
Figure 4.63  Failure of box- beam DP1
Figure 4.64  Failure of box-beam DN2
Figure 4.65  Failure of box- beam LP3
Figure 4.66  Load versus strain for bonded prestressing tendons
CHAPTER 5
DESIGN APPROACH AND EXAMPLE

In this chapter, flexural and shear design approaches are presented along with design examples in two separate sections. The flexural design approach is based on unified design approach (Grace and Singh, 2002), which takes into account the prestressing and non-prestressing tendons arranged in vertically distributed layers with any material combination. Design examples are presented for easy understanding of the approach and for the experimental evaluation of the design equations. The flexural and shear design approaches are presented in the following sections:

5.1 Flexural Design Approach

Following are the basic steps to be followed for the flexural design of prestressed beam with bonded and unbonded CFRP tendons/strands.

1. Compute the required moment capacity

Let the dead load moment = $M_D$, and Live load moment = $M_L$

Required moment capacity of the beam,

$$M_{\text{required}} = 1.4 M_D + 1.7 M_L$$


2. Select and proportion the cross-section

Select and proportion the cross-section of the beam and specify the number and arrangement of the bonded prestressing tendons, unbonded post-tensioning tendons, and non-prestressing tendons in tension and compression zones. The maximum prestress force in the CFRP tendons should be limited to 65% of the specified tensile strength of tendons (JSCE, 1997 and Bakht et al., 2000).
3. Compute balanced ratio ($\rho_b$)

The balanced ratio as expressed below is based on strain compatibility in the cross-section and signifies the reinforcement ratio at which simultaneous failure of concrete in compression and rupture of the bottom bonded prestressing tendons occurs. This balanced ratio (Burke and Dolan, 2001) is based on four basic assumptions: (a) the ultimate compression strain ($\varepsilon_{cu}$) is 0.003, (b) the nonlinear behavior of concrete is modeled using an equivalent rectangular stress block, (c) tendon failure occurs at the ultimate tensile strain of tendon, $\varepsilon_{fu}$, and (d) equivalent prestressing tendon is located at the centroid of multiple layers of prestressing tendons of the same material properties.

$$\rho_b = 0.85 \beta_1 \frac{f^{c'}_c}{f_{fu}} \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu} - \varepsilon_{pbmi}}$$

(5.1)

where

$\beta_1 = \text{factor defined as the ratio of the depth of equivalent rectangular stress block to the distance from the extreme compression fiber to the neutral axis}$

$f^{c'}_c = \text{specified compressive strength of concrete}$

$f_{fu} = \text{specified tensile strength of bonded prestressing tendons}$

$\varepsilon_{pbmi} = \text{initial prestressing strain in bonded prestressing tendons of m$^\text{th}$ row (bottom row)}$

It is to be noted that the above balanced ratio is based on material properties of bonded prestressing tendons with the assumption that bonded prestressing tendons are susceptible to early failure. Here, the actual reinforcement ratio of the section is defined as the ratio of total weighted cross-sectional area of tendons to the effective concrete cross-sectional area. Weighting factor ($\alpha_i$) is defined as ratio of the stress in a particular equivalent tendon (i.e., a tendon located at the centroid of tendons of the same material and having the cross-sectional area equal to the total cross-sectional area of
corresponding tendons) at the balanced condition to the specified ultimate strength of bonded pretensioning tendons. Here, balanced condition refers to condition at which crushing of concrete and rupture of bottom pretensioning tendons occurs simultaneously. The following expression should be used for calculating the reinforcement ratio. This expression is obtained using equilibrium of forces and compatibility of strains in the cross-section.

\[
\rho = \frac{\sum_{i=1}^{p} \alpha_i A_{fi}}{bd_m} \quad (5.2)
\]

where

\[
\alpha_i = \frac{f_{bi}}{f_{fu}} \quad (5.3)
\]

\(A_{fi}\) = cross-sectional area of reinforcement of a particular material (\(A_{fi}\) is positive for tensile reinforcement and negative for compression reinforcement)

\(b\) = flange width of the beam

\(f_{bi}\) = total stress in an equivalent tendon of a specific material at the balanced condition.

\(p\) = total number of reinforcing materials

\(d_m\) = distance of centroid of bottom prestressing tendons from the extreme compression fiber

4. Compute cracking moment (\(M_{cr}\)) of section

The cracking moment can be found using the concept that stress in the extreme tensile fiber of the prestressed section under the superimposed moment (equal to cracking moment) should be equal to the modulus of rupture (\(f_r\)) of concrete (Lin and Burns, 1981).

\[
M_{cr} = (f_r + \sum \sigma_{bp}) S_b \quad (5.4)
\]
where

\[ f_r = 6.0 \sqrt{f_c} \text{ psi} \]  

\[(\text{ACI 318, 2002})\]

where \( f_c \) is 28-day strength of concrete in psi units.

\[ \sum \sigma_{bp} = \text{resultant stress at extreme tension fiber of the beam due to effective pretensioning and post-tensioning forces.} \]

\( S_b = \text{section modulus corresponding to extreme tension fiber} \)

5. Calculate flexural capacity

The flexural capacity of box-beam provided with prestressing bonded and unbonded tendons arranged in vertically distributed layers (Figure 5.1) and classified as significantly under reinforced, under-reinforced, and over-reinforced beams can be determined using the following approach.

A. Significantly under-reinforced beams

The reinforcement ratio (\( \rho \)) of significantly under-reinforced beams lies below \( 0.5 \rho_b \) (Burke and Dolan, 2001). The failure of beams will occur due to rupture of the bottom prestressing tendons. The compressive stress distribution at ultimate for a typical significantly under-reinforced box-beam will be triangular (Figure 5.2) because compressive stresses in such section will be within linear elastic range. The strain in the bottom prestressing tendon will be equal to the specified rupture strain for prestressing tendons.

Assume that there are ‘m’ rows of pretensioning tendons and ‘k’ rows of non-prestressing bars arranged vertically in the box-beam sections. The first row lies at the top, while the \( m^{th} \) and \( k^{th} \) rows lie near the bottom of beams. Figure 5.1 shows a typical box-beam section provided with bonded and unbonded tendons in vertically distributed layers.
The depth to neutral axis is defined as \( n = k_u d_m \), where coefficient \( k_u \) for the significantly under-reinforced section is defined by Eq. (5.5). This equation is derived using compatibility and equilibrium equations

\[
k_u = \frac{-B+\sqrt{B^2-4AC}}{2A}
\]

(5.5)

where

\[
A = \frac{b d_m^2}{2} f_u \left(1 - \frac{f_{pmi}}{f_u}\right) \frac{E_c}{E_f}
\]

\[
B = \left[F_{pi} d_m + \varepsilon_f d_m \left(\sum_{i=1}^{q} A_{fi} E_{fi} + \Omega_c A_{fu} E_{fp}\right)\right]
\]

\[
C = -\left[F_{pi} d_m - \varepsilon_f \left(\sum_{i=1}^{q} A_{fi} E_{fi} h_i + \Omega_c A_{fu} E_{fp} d_u\right)\right]
\]

\( A_{fi} \) = cross-sectional area of bonded tendons in a particular layer (it is negative for tendons in compression zone)

\( b \) = flange width of beam

\( d_m \) = distance from the extreme compression fiber to the centroid of the bottom prestressing tendons

\( F_{pi} \) = total initial effective pre-tensioning and post-tensioning force

\( E_{fi} \) = modulus of elasticity of bonded tendons in a particular layer

\( h_i \) = distance of bonded tendons of an individual layer from the extreme compression fiber

\( \varepsilon_f \) = difference in ultimate rupture strain and initial prestressing strain of bottom prestressing tendons

\( q \) = number of layers of bonded tendon
Following steps are taken to compute the moment carrying capacity of the beam.

i. **Compute strains in bonded tendons and concrete**

Strain in prestressing tendons of an individual row, $\varepsilon_{pbi} = (\varepsilon_{fu} - \varepsilon_{pbi}) \left( \frac{d_j - n}{d_m - n} \right) + \varepsilon_{pbi}$

(for $j = 1, m$)

Strain in non-prestressing bars of an individual row, $\varepsilon_{pni} = (\varepsilon_{fu} - \varepsilon_{pbi}) \left( \frac{h_j - n}{d_m - n} \right)$ (for $j = 1, k$)

Strain in non-prestressing bars of flange top, $\varepsilon_{pnt} = (\varepsilon_{fu} - \varepsilon_{pbi}) \left( \frac{n - d_t}{d_m - n} \right)$

Strain in non-prestressing bars of flange bottom, $\varepsilon_{pnb} = (\varepsilon_{fu} - \varepsilon_{pbi}) \left( \frac{n - d_b}{d_m - n} \right)$

Strain in concrete at the extreme compression fiber, $\varepsilon_{c} = (\varepsilon_{fu} - \varepsilon_{pbi}) \left( \frac{n}{d_m - n} \right)$

where

$d_j$ = depth from extreme compression fiber to the centroid of prestressing tendons of an individual row

$h_j$ = depth from extreme compression fiber to the centroid of non-prestressing bars of an individual row

$h_k$ = depth from extreme compression fiber to the centroid of bottom non-prestressing bars

$d_t$ = depth from extreme compression fiber to the centroid of compressive non-prestressing bars at the bottom of flange
\[ d_t = \text{depth from extreme compression fiber to the centroid of non-prestressing bars at the top of the beam-section} \]

\[ n = \text{depth to the neutral axis from the extreme compression fiber} \]

ii. **Compute strain in unbonded post-tensioning tendons**

Strain in the unbonded post-tensioning tendons, \( \varepsilon_{pu} = \varepsilon_{pui} + \Delta \varepsilon_{pu} \)

\[ \Delta \varepsilon_{pu} = \Omega_c \frac{(\epsilon_{fu} - \epsilon_{pmb})(d_u - n)}{(d_m - n)} \]

where

\[ \Omega_c = \text{bond reduction coefficient for elastic cracked section (Naaman and Alkhairi, 1991)} \]

\[ = \frac{I_{cr}}{I_{tr}} \]

\[ \Omega = \text{bond reduction coefficient for elastic uncracked section (Naaman and Alkhairi, 1991)} \]

\[ = \frac{1}{2} \text{ for one-point loading} \]

\[ = \frac{2}{3} \text{ for two-point loading or uniform loading} \]

\[ I_{tr} = \text{gross transformed moment of inertia of cross-section} \]

\[ I_{cr} = \text{transformed moment of inertia of cracked section} \]

It is to be noted that bond reduction coefficients are introduced to take into account the lower strain in unbonded tendons with respect to equivalent bonded tendons (Naaman and Alkhairi, 1991). Also, it is assumed that the section is not cracked under the service load condition.
iii. **Compute stresses in tendons**

Stress in bonded prestressing tendons of an individual row, \( f_{pbj} = E_f \times \varepsilon_{pbj} \leq f_{fu} \)

Stress in unbonded post-tensioning tendons, \( f_{pu} = E_{fp} \times \varepsilon_{pu} \leq f_{fup} \)

Stress in non-prestressing bars of an individual row, \( f_{pnj} = E_{fn} \times \varepsilon_{pnj} \leq f_{fun} \)

Stress in non-prestressing bars at flange top, \( f_{pnt} = E_f \times \varepsilon_{pnt} \leq f_{fu} \)

Stress in non-prestressing bars at flange bottom, \( f_{pnb} = E_f \times \varepsilon_{pnb} \leq f_{fu} \)

iv. **Compute resultant forces in tendons**

Resultant force in bonded prestressing tendons of each row, \( F_{pbj} = f_{pbj} \times A_{fb} \)

Resultant force in unbonded post-tensioning tendons, \( F_{pu} = f_{pu} \times A_{fu} \)

Resultant force in non-prestressing tendons of each row, \( F_{pnj} = f_{pnj} \times A_{fn} \)

Resultant force in non-prestressing bars at flange top, \( F_{pnt} = f_{pnt} \times A_{fnt} \)

Resultant force in non-prestressing bars at flange bottom, \( F_{pnb} = f_{pnb} \times A_{fnb} \)

where

\( A_{fb} = \) total cross-sectional area of bonded prestressing tendons in each row

\( A_{fn} = \) total cross-sectional area of non-prestressing bars in each row

\( A_{fnt} = \) total cross-sectional area of compression non-prestressing bars at flange top

\( A_{fnb} = \) total cross-sectional area of compression non-prestressing bars at flange bottom

\( A_{fu} = \) total cross-sectional area of unbonded tendons

v. **Compute ultimate moment carrying capacity**

Let the centroid of the resultant \( (F_R) \) of tensile forces \( F_{pbj} \) (for \( J = 1, m \)), \( F_{pnj} \) (for \( J = 1, k \)), and \( F_{pu} \) lies at a distance, \( d \), and the centroid of resultant compression force \( (C) \) lies at a distance \( \overline{d} \) from the extreme compression fiber. Nominal moment of resistance \( (M_n) \) is expressed as below:

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\[ M_n = F_R (d-d) \]  

(5.6)

where

\[ F_R = \sum_{j=1}^{m} F_{pblj} + \sum_{j=1}^{k} F_{pmj} + F_{pu} \]  

(5.7)

Design moment capacity, \( M_u \), is expressed as follow:

\[ M_u = \phi M_n \]  

(5.8)

where

\[ \phi = \text{strength reduction factor; } \phi = 0.85 \text{ for CFRP tendons (Burke and Dolan, 2001 and CHBDC, 2000)} \]

vi. Compare design moment capacity and required moment capacity

\[ M_u \geq M_{\text{required}}, \text{ i.e., } \phi M_n \geq M_{\text{required}} \]  

(5.9)

vii. Check for stresses in concrete

\[ f_c = f_{c0} \left( \frac{f_{pblm}}{f_{fu}} \right) \left( \frac{n}{d_m-n} \right) \frac{E_c}{E_f} \]  

(5.10)

\[ E_c = \text{modulus of elasticity of concrete} \]

\[ f_{pblm} = \text{initial effective prestress in the bottom tendons} \]

\[ f_c = \varepsilon_c \times E_c \leq 0.40 f_c' \]  

(Tan et al., 2001)  

(5.11)

where \( E_c \) can be taken as equal to 57000 \( \sqrt{f_c'} \) psi (ACI 318, 2002) in the absence of experimental results. If \( f_c \geq 0.40 f_c' \), then moment capacity of beam should be calculated as per the procedure outlined below for under-reinforced beam.
B. Under-reinforced beams

The reinforcement ratio ($\rho$) of under-reinforced beams lies between 0.5 $\rho_b$ and $\rho_b$ (Burke and Dolan, 2001). In this class of beams, the rupture of bottom prestressing tendons governs failure. However, unlike significantly under-reinforced beams, the stress in the concrete at failure of under-reinforced beams will be within the non-linear range. The concrete stress distribution can be approximated by Whitney’s rectangular stress block. The strain distribution will be similar to that for the significantly under-reinforced beam. Figure 5.3 shows the strain and stress distributions for a typical under-reinforced box-beam section. The depth to the neutral axis ($n = k_u d_m$) of under-reinforced beam can obtained using coefficient ($k_u$) defined by Eq. (5.12).

$$k_u = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A} \quad (5.12)$$

where

$$A = 0.85 f'_c b \beta_1 d_m^2$$

$$B = -[A + F_{pi} d_m + \varepsilon_f d_m (\sum_{i=1}^{q} A_{fi} E_{fi} + \Omega_u A_{fu} E_{fp})]$$

$$C = [F_{pi} d_m + \varepsilon_f (\sum_{i=1}^{q} A_{fi} E_{fi} h_i + \Omega_u A_{fu} E_{fp} d_u)]$$

i. Compute strains and stresses

The strains, stresses, and forces in bonded tendons are calculated in the same manner as for significantly under-reinforced beams. However, computation of strains and stresses in the unbonded tendon is based on the ultimate bond reduction coefficient ($\Omega_u$) due to non-linearity of stress and strain relationship in the concrete. The resultant compressive force in concrete is computed using Whitney’s equivalent rectangular stress block. The compression force resultants, $C_f$, are expressed as follows:

$$C_f = 0.85 f'_c b \beta_1 n \quad (5.13)$$
The ultimate bond reduction coefficient (Naaman and Alkhairi, 1991) is expressed as follows:

\[
\Omega_u = \frac{2.6}{L_u/d_u}
\]

for one-point loading

\[
= \frac{5.4}{L_u/d_u}
\]

for two-point loading or uniform loading

where

\[L_u = \text{Horizontal distance between the ends of the post-tensioning strands.}\]

ii. Compute nominal and design moment capacities

\[M_n = F_R (d - \bar{d}) \quad (5.14)\]

\[M_d = \phi M_n \geq M_{\text{required}} \quad (5.15)\]

C. Over-reinforced beams

For over-reinforced beams, reinforcement ratio \(\rho\) is greater than balanced ratio \(\rho_b\) (Burke and Dolan, 2001). The failure of over-reinforced beams is governed by the crushing of concrete in the compression zone. The stress in the concrete at failure will be in the non-linear range and hence, the stress distribution can be approximated by Whitney’s rectangular stress block. Fig. 5.4 shows a typical distribution of strains, stresses, and stress-resultants for an over-reinforced box-beam section. In over-reinforced sections, the depth to the neutral axis \((n = k_u d_m)\) can be determined using the coefficient \(k_u\) defined in Eq. (5.16). This coefficient is derived using equilibrium and compatibility equations for the section.

\[k_u = \frac{A + \sqrt{A^2 + 4B}}{2} \quad (5.16)\]

where
\[ A = \frac{\left( \sum_{j=1}^{m} A_{fb} E_f \varepsilon_{pbji} + A_{fu} \varepsilon_{pu} E_{fp} \right) - \varepsilon_{cu} \left( \sum_{j=1}^{q} A_{fj} E_{fj} + \Omega_u A_{fu} E_{fp} \right)}{0.85 f_c' b \beta_1 d_m} \]

\[ B = \frac{\varepsilon_{cu} \left( \sum_{j=1}^{q} A_{fj} E_{fj} h_j + \Omega_u A_{fu} E_{fp} d_u \right)}{0.85 f_c' b \beta_1 d_m^2} \]

\( A_{fb} = \) cross-sectional area of prestressing or non-prestressing bonded tendons in an individual row

\( E_{fj} = \) modulus of elasticity of bonded tendons of an individual row

\( h_j = \) distance of bonded tendons from the extreme compression fiber

\( q = \) total number of layers of bonded prestressing and non-prestressing tendons

i. Compute strains in tendons

Strain in bonded prestressing tendons, \( \varepsilon_{pbji} = \frac{0.003}{n} (d_j - n) + \varepsilon_{pbji} \) (for \( j = 1, m \))

Strain in non-prestressing bars, \( \varepsilon_{pnj} = \frac{0.003}{n} (h_j - n) \) (for \( j = 1, k \))

Strain in non-prestressing bars at flange top, \( \varepsilon_{pnt} = \frac{0.003}{n} (n - d_t) \)

Strain in non-prestressing bars at flange bottom, \( \varepsilon_{pnb} = \frac{0.003}{n} (n - d_b) \)

Strain in unbonded tendons, \( \varepsilon_{pu} = \varepsilon_{pui} + \Delta \varepsilon_{pu} \)

\[ = \varepsilon_{pui} + \Omega_u \frac{0.003}{n} (d_u - n) \]
It is to be noted that the stresses and forces in tendons and concrete can be calculated as per equations for under-reinforced beams.

ii. **Compute the nominal moment capacity**

Resultant of the total tensile force, \( F_R = \sum_{j=1}^{m} F_{pbj} + \sum_{j=1}^{k} F_{pnj} + F_{pu} \)

Let the centroids of the resultant tensile force and resultant compression force lie at distances of \( d \) and \( \bar{d} \) from the extreme compression fiber, respectively.

Ultimate nominal moment capacity, \( M_n = F_R (d - \bar{d}) \)

Design flexural capacity, \( M_u = \phi M_n \geq M_{required} \)

**5.1.1 Deflection and Stresses under Service Load Condition**

Assuming that the service live load is applied through four-point loading frame, the maximum beam deflection and tendon stresses prior to cracking load can be computed using the expressions given below:

Deflection due to applied load, \( \delta_a = \frac{M_L L_1^2}{8 E_c I_c} \left[ \frac{8}{3} + 4 \left( \frac{L_2}{L_1} \right) + \left( \frac{L_2}{L_1} \right)^2 \right] \)

Deflection due to dead load, \( \delta_d = \frac{5}{384} \frac{W_d L^4}{E_c I_c} \)

Deflection due to prestressing forces, \( \delta_p = \frac{L^2}{8 E_c I_c} \left[ F_{pre} e_b + F_{post} e_u \left( \frac{L_u}{L} \right) \right] \)

Net downward deflection of the beam, \( \delta = \delta_a + \delta_d - \delta_p \)
Stress in a bonded tendon of individual layer, $f_{pbj} = E_f \epsilon_{pbji} + \frac{E_f}{E_c} \frac{M (d_j - y_{tc})}{I_c}$

Stress in unbonded tendon, $f_{pu} = E_{fp} \epsilon_{pui} + \frac{E_{fp} (M - M_D)}{E_c I_c} \Omega$

Stress in non-prestressing tendons, $f_{pn} = \frac{E_{fn} M (h_j - y_{tc})}{E_c I_c}$

where

$L_1 = \text{distance between support and nearest load point}$

$L_2 = \text{longitudinal distance between two pairs of load points}$

$L = \text{effective span of the beam}$

$M_L = \text{service live load moment}$

$e_b = \text{eccentricity of resultant pretensioning force from the centroid of box-beam section}$

$e_u = \text{eccentricity of unbonded tendons from the centroid of box-beam section}$

$I_c = \text{moment of inertia of composite cross-section of box-beam}$

5.1.2 Nonlinear Response

The response of the beam under service load condition and before cracking can be determined using simple linear elastic beam theory. However, to predict the overall response of the beam from the onset of cracking of the section to the ultimate failure, a nonlinear analysis is required. The non-linear stress and strain relationship for concrete can be modeled by the expression given in Eq. (5.17). Similar parabolic stress strain relationship for concrete was assumed by Tan et al. (2001), except that they have taken $\epsilon_{cu}$ equal to 0.002. However, in the present investigation $\epsilon_{cu}$ has been taken as 0.003 as per the recommendation of ACI 318 (2002).
\[
\frac{f_c}{f'_{c}} = 2.0 \frac{\varepsilon_c}{\varepsilon_{cu}} - \left( \frac{\varepsilon_c}{\varepsilon_{cu}} \right)^2 \quad (5.17)
\]

where

\( f_c \) = stress in concrete corresponding to strain \( \varepsilon_c \)

The resultant compressive force in concrete based on a nonlinear stress-strain relation can be computed using equivalent rectangular stress block factors at any load stage. The stress block factors can be determined by equating the resultant compression force and its location obtained from non-linear stress-strain relation to that obtained by equivalent stress block. Eq. (5.18) expresses the resultant compression force, while Eq. (5.19) can be used to locate the centroid of resultant compression force.

\[
\int_{0}^{n} f_c b \, dy = \alpha f_c' \beta n \quad b \quad (5.18)
\]

\[
\frac{\int_{0}^{n} f_c b \, dy}{y_c} = \frac{\int_{0}^{n} f_c b \, dy}{\int_{0}^{n} f_c b \, dy} \quad (5.19)
\]

Using Eqs. (5.17), (5.18), and (5.19), the stress block factors for rectangular and flanged sections can be obtained using following expressions:

The stress block factors \( (\alpha_1, \beta_1) \) are evaluated using Eq. (5.20a) and Eq. (5.20b).

\[
\alpha_1 \beta_1 = \frac{\varepsilon_t}{\varepsilon_{cu}} \frac{1.0}{3.0} \left( \frac{\varepsilon_t}{\varepsilon_c} \right)^2 \quad (5.20a)
\]

\[
\beta_1 = \frac{4.0 - \frac{\varepsilon_t}{\varepsilon_{cu}}}{6.0 - \frac{2 \varepsilon_t}{\varepsilon_{cu}}} \quad (5.20b)
\]
where
\[ \varepsilon_t = \text{strain at the extreme compression fiber at a specific stage of loading} \]

Using linear elastic theory for computation of linear response and non-linear stress-strain relation of concrete for non-linear response, a computer program was developed for predicting the overall load versus deflections, strains, stresses, forces in bonded and unbonded tendons, and moment curvature relationships. The non-linear response is based on the incremental strain controlled approach, wherein equilibrium of forces is achieved for each level of compressive concrete strain and corresponding tensile strain. The calculation of deflection is based on numerical integration of curvatures along the length of beam. Figure 5.5 shows the flow chart diagram for predicting the linear and nonlinear responses of box-beams.

### 5.1.3 Experimental Verification

The comparison of the analytical load versus deflection, post-tensioning forces in unbonded tendons, and extreme fiber compressive strains at midspan with corresponding experimental results, obtained from testing of the box-beam (LP3), is presented in Figures 5.6 to 5.8, respectively. It is observed that the analytical and experimental load versus deflection (Figure 5.6), post-tensioning force (Figure 5.7), and extreme fiber compressive strain (Figure 5.8), relationships are in fair agreement. The values of theoretical and experimental cracking loads are almost the same. The cracking and ultimate loads of the box-beams are marked in the figures.

A slight difference in the experimental and analytical results occurred in the advanced post-cracking stage of deformation, and that is due to the experimental loading and unloading cycles applied on the box-beams prior to ultimate load test. The percentage difference in the experimental and analytical ultimate load carrying capacities of box-beam LP3 is about 5%. Similar comparative responses (Figs. 5.9 to 5.11) were obtained for beam DP1 also with almost negligible difference in the analytical and experimental load carrying capacities of the box-beams.

As shown in Figures 5.12 and 5.13, the difference in the experimental and analytical responses of box-beam DN2 (beam without prestress in the unbonded post-tensioning tendons) is also not significant. However, this difference in the analytical and
experimental responses of box-beam DN2 is slightly larger in comparison to the corresponding responses of box-beams DP1 and LP3. The maximum effect of loading and unloading cycles on the difference in the experimental and analytical responses of box-beams is observed on the load versus strain relationships (Figure 5.14) of box-beam DN2.

5.1.4 Parametric Study

To examine the effect of the level of pretensioning and post-tensioning forces on the flexural response and ultimate load carrying capacity of box-beams, the calculated load versus deflection responses of box-beams, prestressed and reinforced with DC1 tendons, are presented in Figs. 5.15 and 5.16. It is observed from Fig. 5.15 that for a particular level of prestressing forces in unbonded post-tensioning tendons (upl = 0.7), the level of pretensioning forces (bpl) has significant effect on the load deflection response of the beam.

From Figure 5.15, it is also observed that variation in the level of pretensioning forces will result in different reinforcement and balanced ratios of the box-beam provided with same pretensioning tendons and non-prestressing rods. The ratio of reinforcement and balanced ratios along with ultimate load carrying capacity of box-beam is shown in the figure for different levels of pretensioning forces (bpl).

It must be noted that the beam with \( \frac{\rho}{\rho_b} \) greater than 1.0 failed due to crushing of concrete while those with \( \frac{\rho}{\rho_b} \) less than 1.0 failed due to rupture of pretensioning tendons. It is shown that pretensioning force level (bpl) of 0.4 resulted in the load carrying capacity (86 kips) of box-beam identical to that for pretensioning force level (bpl) equal to 0.5. However, this level of pretensioning force (bpl equals to 0.4) resulted in lower deflection corresponding to the ultimate failure load in comparison to that for pretensioning force level, bpl equal to 0.5. The difference in the ultimate load responses for bpl equal to 0.4 and 0.5 is attributed to the almost balanced-section behavior of box-beam for bpl equal to 0.4 and under-reinforced box-beam behavior for bpl equal to 0.5. The increase in the bpl beyond 0.5 reduces the load carrying capacity of box-beam. The
lowest load carrying capacity of box-beam was observed for bpl equal to 0.7 due to significantly under-reinforced box-beam behavior at this level of pretensioning forces.

From Fig. 5.16, it is observed that for a particular load, reinforcement ratio ($\rho = 0.0034$), and for a constant level of pretensioning force (bpl = 0.5), deflection is higher for the lower unbonded post-tensioning prestress level in comparison to that for higher level of post-tensioning forces. The higher level of unbonded post-tensioning also results in the higher load carrying capacity of box-beams provided unbonded tendons remain intact at the ultimate failure of the box-beam.

5.2 Flexural Design Example

Problem:

Determine the flexure capacity of a box-beam prestressed with 6 unbonded and 7 bonded Diversified Composites, Inc. (DCI) CFRP tendons and reinforced with 2 non-prestressing bars at bottom and 7 non-prestressing bars at the top. The cross-section details of the box-beam are shown in Figure 5.17. The effective span of the box-beam is 15 ft. The specified tensile strength and maximum percentage elongation of DCI tendons are 280 ksi and 0.0147, respectively. The modulus of elasticity of DCI tendons are 19,000 ksi. All DCI tendons and bars are of 0.374 in. diameter. The properties of concrete are listed below.

Concrete properties:

\[
f'_c = 7,000 \text{ psi (7.0 ksi)}
\]

\[
E_c = 57,000 \sqrt{f'_c} = 4.77 \times 10^6 \text{ psi (4,770 ksi)}
\]

\[
f_r = 6 \sqrt{f'_c} = 500 \text{ psi (0.5 ksi)}
\]

Section properties:

Diameter of bars and tendons, d = 0.374 in.
Area of bars and tendons, A = 0.11 in.$^2$
Width of box beam, b = 38 inches
Moment of inertia of section, I = 5,344 in.$^2$
d_{in} = 10 \text{ in.}

**Prestressing forces**

Initial pretensioning force in each tendon = 20.8 kips
Initial post-tensioning force in each unbonded tendon = 20.9 kips
Total initial pretensioning force in pretensioning tendons = \( 7 \times 20.8 = 145.6 \) kips
Total initial post-tensioning force in unbonded tendons = \( 6 \times 20.9 = 125.4 \) kips
Based on experimental results, the loss in the pretensioning forces is assumed to be equal to 13.6%, whereas corresponding loss in the forces in the unbonded post-tensioning tendon is assumed to 2%.

Initial strain in the bonded pretensioning tendons,
\[ \varepsilon_{pbmi} = \frac{0.864 \times 20.8}{0.11 \times 19,000} = 0.0086 \]

Total effective pretensioning force = \( 0.864 \times 145.6 = 125.8 \) kips
Total effective post-tensioning force = \( 0.98 \times 125.4 = 122.89 \) kips
Total initial effective prestressing force, \( F_{pi} = 248.69 \) kips

**Solution:**

Balanced ratio

\[ \rho_b = 0.85 \beta_1 \frac{f'_c}{f_{fu}} \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu} - \varepsilon_{pbmi}} \]

\( \beta_1 = 0.7 \)
\( f'_c = 7.0 \text{ ksi} \)
\( f_{fu} = 280 \text{ ksi} \)
\( \varepsilon_{cu} = 0.003 \)
\( \varepsilon_{fu} = 0.0147 \)
\( \varepsilon_{pbmi} = 0.0086 \)
\[
\rho_b = 0.85 \times 0.7 \times \frac{7}{280} \times \frac{0.003}{0.003 + 0.0147 - 0.0086} = 0.0049
\]

\[
\sum_{i=1}^{p} A_i \alpha_i \quad \rho = \frac{\sum_{i=1}^{p} A_i \alpha_i}{b \ d_m}
\]

\[b = 38 \text{ in.}\]

\[d_m = 10\]

Figure 5.18 shows the balanced section and the strains induced in the tendons and bars. The stresses in the tendons and bars are calculated accordingly and shown below.

Stress in non-prestressing bars provided at bottom of box-beams = 115.9 ksi

Stress in pretensioning tendons = 280 ksi

Initial strain in unbonded post-tensioning tendons = \[
\frac{0.98 \times 20.9}{0.11 \times 19,000} = 0.0098
\]

Total strain in each unbonded tendon at balanced condition = 0.0098 + 0.001 = 0.0108

Stress in unbonded post-tensioning tendons = \[19,000 \times 0.0108 = 205.2\text{ ksi}\]

Stress in compression non-prestressing bars = 22.8 ksi

\[
\therefore \rho = \frac{1.2821}{38 \times 10} = 0.0034
\]

\[\rho\] is slightly less than \(\rho_b\) and greater than 0.5\(\rho_b\), thus the section is under reinforced.

**Cracking moment**

\[
M_{cr} = (f_r + \Sigma \sigma_{bp}) s_b
\]

\[
\Sigma \sigma_{bp} = \frac{122.89}{360} + \frac{122.89 \times 2 \times 6}{5344} + \frac{125.8}{360} + \frac{125.8 \times 4 \times 6}{5344} = 1.5318 \text{ ksi}
\]

\[
M_{cr} = (0.5 + 1.5318) \times 890.67
\]
= 1809.66 kip-in. = 150.81 kip-ft

**Flexural capacity**

\[ n = k_u d_m \]

\[ k_u = \frac{-B \pm \sqrt{B^2 - 4AC}}{2 A} \]

\[ A = 0.85 \ f'_c \ b \beta_1 \ d_m^2 = 0.85 \times 7 \times 38 \times 0.7 \times 10^2 = 15,827 \]

\[ B = - \left[ A + F_{pi} \ d_m + \varepsilon_f \ d_m \left( \sum_{i=1}^{q} A_{fi} \ E_{fi} + \Omega_u \ A_{fu} \ E_{fp} \right) \right] \]

\[ F_{pi} = 248.69 \text{ kips} \]

\[ d_m = 10 \text{ in.} \]

\[ \varepsilon_f = 0.0061 \]

\[ \sum_{i=1}^{q} A_{fi} \ E_{fi} = 0.77 \times 19,000 + 0.22 \times 19,000 - 0.77 \times 19,000 = 4,180 \]

\[ \Omega_u = \frac{5.4}{\left( \frac{L_u}{d_u} \right)} \]

\[ L_u = 192 \text{ in.} \]

\[ d_u = 8 \text{ in.} \]

\[ \Omega_u = \frac{5.4}{\left( \frac{192}{8} \right)} = 0.225 \]

\[ A_{fu} = 0.66 \]

\[ E_{fp} = 19,000 \text{ ksi} \]

\[ B = - \left[ 15,827 + 248.69 \times 10 + 0.0061 \times 10 \times (4,180 + 2,821.5) \right] = -18741.0 \]

\[ C = \left[ F_{pi} \ d_m + \varepsilon_f \left( \sum_{i=1}^{q} A_{fi} \ E_{fi} \ h_i + \Omega_u \ A_{fu} \ E_{fp} \ d_u \right) \right] \]

\[ F_{pi} \ d_m = 2,486.9 \]

\[ \varepsilon_f = 0.0061 \]
\[ \sum_{i=1}^{q} A_{f_i} E_{f_i} h_i = 0.77 \times 19,000 \times 10 + 0.22 \times 19,000 \times 10 - 0.77 \times 19,000 \times 2 = 1,58,840 \]

\[ \Omega_u A_{fu} E_{fp} d_u = 0.225 \times 0.66 \times 19,000 \times 8 = 22,572 \]

\[ C = 2,486.9 + 0.0061 (1,58,840 + 22,572) = 3,593.5 \]

\[ k_u = 0.241 \]

\[ n = k_u d_m = 0.241 \times 10 = 2.41 \text{ in.} \]

Strain in prestressing tendons

\[ \varepsilon_{pb} = \varepsilon_{fu} = 0.0147 \]

Strain in bottom non-prestressing bars

\[ \varepsilon_{pa} = (0.0147 - 0.0086) = 0.0061 \]

Strain in top non-prestressing bars,

\[ \varepsilon_{pnt} = 0.0061 \left( \frac{2.41 - 2}{10 - 2.41} \right) = 0.00033 \]

\[ \Delta \varepsilon_{pu} = 0.225 \times 0.0061 \times \frac{(8 - 2.41)}{(10 - 2.41)} = 0.001 \]

Strain in the unbonded prestressing tendons \((\varepsilon_{pu}) = \varepsilon_{pui} + \Delta \varepsilon_{pu} \)

\[ = 0.0098 + 0.001 \]

\[ = 0.0108 \]

Stress in bonded prestressing tendons, \(f_{pb} = 280 \text{ ksi} \)

Stress in bottom non-prestressing bars, \(f_{pn} = 19,000 \times 0.0061 \)

\[ = 115.9 \text{ ksi} < 280 \text{ ksi} \quad \text{OK} \]

Stress in unbonded post-tensioning tendons, \(f_{pu} = 19,000 \times 0.0108 \)

\[ = 205.2 \text{ ksi} < 280 \text{ ksi} \quad \text{OK} \]

Stress in top non-prestressing bars, \(f_{pnt} = 0.0061 \times 19,000 = 6.27 \text{ ksi} < 280 \text{ ksi} \quad \text{OK} \]

Resultant force in bonded prestressing tendons, \(F_{pb} = 280 \times 0.77 = 215.6 \text{ kips} \)
Resultant force in bottom non-prestressing bars, \( F_{pn} = 115.9 \times 0.22 = 25.50 \) kips

Resultant force in unbonded post-tensioning tendons, \( F_{pu} = 205.2 \times 0.66 = 135.43 \) kips

Resultant force in top non-prestressing bars, \( F_{ptn} = 6.27 \times 0.77 = 4.83 \) kips

**Nominal Moment Capacity of Box-Beam**

Resultant Compression Force, \( C_R = 0.85 \times 7 \times 38 \times 0.7 \times 2.41 + 4.83 = 386.3 \) kips

Resultant tension force \( F_R = 215.6 + 135.43 + 25.50 = 376.53 \) kips \( \cong C_R \) O.K.

Distance of centroid of resultant compression force from the extreme compression fiber,

\[
\bar{d} = \frac{381.43 \times 0.7 \times 2.41 + 4.83 \times 2}{386.3} = 0.86 \text{ in.}
\]

Distance of the centroid of resultant tension force from the extreme compression fiber, \( d = 10 \) in.

Nominal moment capacity of box-beam, \( M_n = F_R (d - \bar{d}) \)

\[
= 376.53 \times (10-0.86)
\]

\[
= 3,441.5 \text{ kip-in.} = 286.8 \text{ kip-ft}
\]

Dead load of the beam \( W_d = 0.375 \text{ kip/ft} \)

Dead load moment, \( M_D = \frac{0.375 \times 15^2}{8} = 10.55 \text{ kip-ft} \)

Experimental value of the failure load of the beam = 86 kips

Experimental moment capacity of box-beam = \( 0.5 \times 86 \times \frac{80}{12} + 10.55 \)
Percentage difference in the nominal and experimental moment capacities of the box-beam = \[
\frac{(297.2 - 286.8)}{297.2} \times 100 = 3.5 \%
\]

The theoretical nominal moment capacity of box-beam is 3.5% lower than the corresponding experimental value. Thus, the unified design equations (Grace and Singh, 2002) reasonably estimate the nominal moment capacity of the box-beam. It must be noted that the above difference in the analytical and experimental moment capacities could be further reduced by taking into account the parabolic stress-strain relationship for concrete as described earlier in section entitled “Nonlinear Response”.

5.3 Shear Design Approach

In this section, an approach for shear design of CFRP reinforced and prestressed box-beam is presented. In this approach, the nominal shear strength contribution of prestressed concrete is based on ACI 318 (2002) provision by considering flexure-shear cracking and web shear cracking. However, shear strength contribution of CFRP stirrups is based on empirical design equations formulated using experimental results of the present study. The steps to compute nominal shear strength of box-beam reinforced with CFRP stirrups and bars, and prestressed using bonded pretensioning and unbonded post-tensioning tendons are outlined below:

1. **Calculate the factored maximum shear force due to superimposed dead and live loads.**

Let the shear force due dead load = \( V_D \), and shear force due to live load = \( V_L \)

Factored shear force,

\[
V_u = 1.4 V_D + 1.7 V_L
\]  
\text{(ACI 318, 2002)}  \quad (5.21)
2. **Compute the nominal shear strength contribution of concrete**

The shear strength contribution of concrete can be computed by considering two typical modes of failure, i.e., web-shear cracking and flexure-shear cracking.

**(a) Web-shear cracking:** This mode of failure occurs in the zone where shear force is predominant and bending moment is negligible. The shear strength contribution of the concrete corresponding to this mode of failure can be computed by the following expression (ACI 318, 2002):

\[ V_{cw} = b_w d (3.5 \sqrt{f'_c} + 0.3 f_{cc}) + V_p \]  \hspace{1cm} (5.22)

where

- \( b_w = \) web width of the section
- \( d = \) distance from the extreme compression fiber to the centroid of CFRP reinforcement
- \( f_{cc} = \) compressive stress at the centroid of the concrete due to effective prestress force
- \( V_p = \) vertical component of the prestress.

**(b) Flexure-shear cracking:** This mode of failure occurs in the region where combined effects of shear and bending moment are appreciable. The shear contribution of concrete corresponding to this mode of failure is given by the following expression:

\[ V_{ci} = 0.6 \sqrt{f'_c} b_w d + V_o + \frac{V_i}{M_{max}} M_{cr} \geq 1.7 \sqrt{f'_c} b_w d \]  \hspace{1cm} (5.23)

where \( V_i \) and \( M_{max} \) are respectively the shear and bending moments at the section under consideration. These parameters can be calculated with or without load factor, since ratio \( \frac{V_i}{M_{max}} \) remains unchanged.

\( M_{cr} = \) Moment causing flexural cracking, computed by Eq. (5.24).
\[ M_{cr} = \frac{I}{c_2} (6\sqrt{f_c^e} + f_{2p} - f_o) \]  

(5.24)

\( V_o \) = shear force due to self-weight of the beam and is computed without load factor

\( f_o \) = flexural stress in the concrete at the bottom face of the beam due to self weight and is calculated without load factor

\( c_2 \) = distance from the concrete centroid of the section to the bottom face

\( I \) = moment of inertia of the concrete cross-section

\( f_{2p} \) = concrete compressive stress at the bottom face resulting from the axial and bending effects of eccentric prestress force

The nominal shear strength of the concrete, \( V_c \) = smaller of \( V_{cw} \) and \( V_{ci} \)

3. **Compute the required spacing of the trial stirrups and nominal shear strength of stirrups**

Let the cross-sectional area of trial stirrup = \( A_{fv} \), then spacing of stirrups can be obtained by the following expression:

\[
s = \frac{\phi A_{fv} f_{fv} d}{V_u - \phi V_c} \]  

(5.25)

where

\( \phi \) = strength reduction factor, which is taken as 0.85 for CFRP bars/tendons

\( f_{fv} \) = effective stress in the stirrups

\( = \frac{s}{0.3 + 0.05} f_{tg} \leq f_u \)

\( f_{tg} \) = guaranteed tensile strength of stirrup bars

\( f_u \) = uni-axial stirrup strength
= [0.0171 \frac{1_d}{d_c} + 0.20 ] \text{ for MIC C-bar stirrups}

If the spacing determined from the trial stirrup size is too close for placement economy or practicality, or if it is so large that the maximum spacing requirements control over too great a part of the beam span, then revised bar size is selected and appropriate spacing is recalculated. Alternatively, if size of stirrup and spacing is known, then nominal shear strength contribution of stirrups can be computed by the following expression.

\[ V_s = \frac{A_{fv} f_{fv}}{s} d \]  \quad (5.26)

4. **Compute the nominal and design shear capacity of the beam**

Nominal shear strength of the beam, \( V_n = V_c + V_s \)  \quad (5.27)

Design shear capacity of the beam, \( V_d = \phi V_n \geq V_u \)  \quad (5.28)

5. **Check for the minimum shear reinforcement**

The minimum area of shear reinforcement to be provided is taken to be equal to the smaller of the following values

\[ A_v = 50 \frac{b_w s}{f_{fv}} \]  \quad (5.29)

\[ A_v = \frac{A_p f_{pu}}{80 f_{fv} d \sqrt{b_w}} \]  \quad (5.30)

where

\[ A_p = \text{cross-sectional area of prestressing tendons} \]

\[ f_{pu} = \text{ultimate tensile strength of the prestressing tendons} \]
The maximum spacing of stirrups should not exceed the smaller of 0.75 \( h \) or 24 in. (ACI 318 provision for prestressed members, 2002). If the value of \( V_s \) exceeds \( 4 \sqrt{f'_c} b_w d \), these limits are reduced by one-half.

### 5.4 Shear Design Example

**Problem**

Evaluate the shear resistance of two box-beams prestressed with 6 un-bonded and 7 bonded CFRP prestressing (DCI) tendons. In addition, each beam is provided with 4 non-prestressing bars at bottom and 7 non-prestressing bars at the top. The cross-sectional details of both beams are the same and are shown in Figure 5.17. The effective span of each beam is 15 ft. Note that Beam-1 is provided with 9.5mm (0.374 in.) CFRP stirrups at center-to-center spacing of \( d/2 \) (\( d \) is effective depth of the beam), while Beam-2 is provided with similar stirrups at center-to-center spacing of \( d/3 \). If the experimental central load carrying capacities of the control beam (beam without shear reinforcement), Beam-1, and Beam-2 are 79.8 kips, 116.1 kips, and 120.4 kips, respectively, then compute the percentage difference in theoretical and experimental shear strengths of beams. Followings are the specified characteristics of stirrups and tendons.

- Modulus of elasticity of CFRP (MIC-C bar) stirrups, \( E_f = 15,794 \text{ ksi} \)
- Guaranteed breaking strength of stirrup bars, \( f_{tg} = 270 \text{ ksi} \)
- Diameter of stirrup bar, \( d_b = 0.374 \text{ in.} \)
- Diameter of each tendon, \( d_b = 0.374 \text{ in.} \)
- Total effective prestressing force in each tendon = 20.79 kips

Following design steps are taken to evaluate the nominal and design shear strengths of prestressed concrete box-beam.
Step 1: Compute the nominal shear strength contribution of concrete

Nominal shear strength contribution of concrete is based on two failure modes (Nilson, 1987).

(a) Web shear cracking: The nominal shear strength contribution of concrete for this failure mode is given by, \( V_{cw} \)

where

\[
V_{cw} = b_w d (3.5 \sqrt{f_c'} + 0.3 f_{cc}) + V_p
\]

Here, \( V_p = 0 \), since prestressing tendon are straight and normal to the applied shear force.

\[
f_{cc} = \frac{F_{pre}}{A} + \frac{F_{post}}{A}
\]

\[
= \frac{7 \times 20.79}{360} + \frac{6 \times 20.79}{360} = 0.404 + 0.347 = 0.751 \text{ ksi}
\]

\[= 751 \text{ psi}\]

Thus,

\[
V_{cw} = 14 \times 9.8 \ (3.5 \sqrt{7400} + 0.3 \times 751) = 72220 \text{ lbs} = 72.22 \text{ kips}
\]

(b) Flexural–shear cracking: The nominal shear strength contribution of concrete for this failure mode is given by, \( V_{ci} \)

where

\[
V_{ci} = 0.6 \sqrt{f_c'} b_w d + V_o + \frac{V_i}{M_{\text{max}}} M_{cr} \geq 1.7 \sqrt{f_c'} b_w d
\]

For the third-point loading, let the applied central load is \( P \).
\[ V_i = \frac{P}{2} \]

\[ M_{\text{max}} = \frac{P \times L}{2} \times \frac{PL}{3} = \frac{PL}{6} \]

\[ \frac{V_i}{M_{\text{max}}} = \frac{3}{L} \]

\[ M_{\text{cr}} = \frac{I}{c_2} (6\sqrt{f_c'} + f_{2p} - f_o) \]

Assuming unit weight of concrete as 150 lbs/ft³.

Cross-sectional area of section, \( A = 38 \times 12 - 2 \times 12 \times 4 = 360 \) in.²

Dead load of the beam/unit length, \( W_D = 150 \times 360/(12 \times 12) = 375 \) lbs/ft

Dead load moment at third-point section = \( \frac{W_D L^2}{9} = \frac{0.375 \times 15^2}{9} = 9.375 \) kip-ft

Moment of inertia of concrete cross-section, \( I = 5344 \) in.⁴

\[ f_o = \frac{9.375 \times 12 \times 6}{5344} = 0.126 \text{ ksi} = 126 \text{ psi} \]

\[ f_{2p} = \frac{F_{\text{pre}} + F_{\text{pre}}c_2}{A} + \frac{F_{\text{post}} + F_{\text{post}}c_2}{A} \]

\[ = \frac{7 \times 20.79 + 7 \times 20.79 \times 3.8 \times 6}{360} + \frac{6 \times 20.79 + 6 \times 20.79 \times 1.8 \times 6}{5344} \]

\[ = 0.404 + 0.621 + 0.347 + 0.252 = 1.624 \text{ ksi} = 1624 \text{ psi} \]

\[ c_2 = 6 \text{ in.} \]

\[ M_{\text{cr}} = \frac{5344}{6} (6\sqrt{7400+1624-126}) = 1793.9 \text{ kip-in.} \]
\[ V_o = \frac{W_{DL}}{6} = \frac{0.375 \times 15}{6} = 0.938 \text{ kips} \]

\[ 1.7 \sqrt{f_c} \ t_{\text{w}} \ d = 1.7 \times \sqrt{7400} \times 14 \times 9.8 = 20.06 \text{ kips} \]

\[ V_{ci} = \frac{0.6 \times \sqrt{7400} \times 14 \times 9.8}{1000} + 0.938 + \frac{3 \times 1793.9}{15 \times 12} = 7.08 + 0.938 + 29.9 = 37.9 \text{ kips} \]

\[ > 20.06 \text{ kips} \]

Since, \( V_{ci} \) is smaller than \( V_{cw} \), hence \( V_c = V_{ci} = 37.9 \text{ kips} \)

Experimental value of the shear strength contribution of concrete = 39.3 kips

Percent difference in the theoretical and experimental values of the nominal shear strength of concrete = \( \frac{39.3 - 37.9}{39.3} \times 100 = 3.6\% \)

Thus, design equations reasonably estimate the nominal shear strength contribution of concrete.

**Step 2: Compute the nominal shear strength contribution of stirrups**

From symmetry, embedded length of C-bar stirrups in box-beam, \( l_d = \frac{7.8}{2} = 3.9 \text{ in.} \)

\[ \frac{l_d}{d_e} = \frac{3.9}{0.35} = 11.1 \]

Uni-axial strength of C-bar stirrups, \( f_u = [0.0171 \frac{l_d}{d_e} + 0.20] f_{ig} = 105.2 \text{ ksi} \)

\[ (a) \text{ Shear strength contribution of stirrups provided at } s = d/2 \]

Effective stirrup stress, \( f_{iv} = [0.3 \frac{s}{d} + 0.05] f_{ig} = 0.2 \times 270 \]

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\[ V_{sd2} = \frac{A_{sv} f_{sv} d}{s} = \frac{0.192 \times 54 \times d}{d/2} = 20.7 \text{ kips} \]

(b) **Shear strength contribution of stirrups provided at } s = \frac{d}{3}\]

\[ f_{sv} = [0.3 \frac{s}{d} + 0.05] f_{ig} = 0.15 \times 270 = 40.5 \text{ ksi} \]

\[ V_{sd3} = \frac{A_{sv} f_{sv} d}{s} = \frac{0.192 \times 40.5 \times d}{d/3} = 23.3 \text{ kips} \]

**Step 3: Compute the nominal shear strength of box-beam**

The nominal shear strength of box-beam is computed as follows:

(a) **Nominal shear strength of box-beam for } s = \frac{d}{2}**

\[ V_n = V_c + V_{sd2} = 37.9 + 20.7 = 58.6 \text{ kips} \]

Experimental shear strength = \( \frac{116.1}{2} = 58.05 \text{ kips} \)

Percent difference = \( \frac{(58.6 - 58.05)}{58.05} \times 100 = 0.9 \% \)

(b) **Nominal shear strength of box-beam for } s = \frac{d}{3}**

\[ V_n = V_c + V_{sd3} = 37.9 + 23.3 = 61.2 \text{ kips} \]

Experimental shear capacity of Beam-2 = \( \frac{120.4}{2} = 60.2 \text{ kips} \)

Percent difference = \( \frac{(61.2 - 60.2)}{60.2} \times 100 = 1.7 \% \)

Thus, it is observed that recommended effective stirrup stress model in conjunction with uni-axial stirrup strength model estimates the shear strength of prestressed box-beam with an acceptable error of less than 2\%.  

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Step 4: Compare the design and experimental shear strength values

(a) Design shear strength of box-beam for \( s = d/2 \)

Design shear strength, \( V_d = \phi V_n = 0.85 \times 58.6 = 49.81 \) kips

Percent difference in design and experimental shear strengths = \( \frac{(58.05 - 49.81)}{58.05} \times 100 \)
\[ = 14.2\% \]

(b) Design shear strength of box-beam for \( s = d/3 \)

Design shear strength, \( V_d = \phi V_n = 0.85 \times 61.2 = 52.02 \) kips

Percent difference in design and experimental shear strengths = \( \frac{(60.2 - 52.02)}{60.2} \times 100 \)
\[ = 13.6\% \]

Thus, on average design shear strength of beams is 13.9 % lower than the actual strength value, which is consistent with strength reduction factor (\( \phi \)) of 0.85.
Figure 5.1 Typical box-beam section with bonded and unbonded tendons
Figure 5.2 Strain and stress distributions for a significantly under-reinforced box-beam at ultimate
Figure 5.3 Strain and stress distributions for an under-reinforced box-beam at ultimate
Figure 5.4 Strain and stress distributions for over-reinforced box-beam at ultimate
Figure 5.5 Flow chart for computation of linear and nonlinear response of prestressed concrete box-beam
Figure 5.6  Load versus deflection relationship for box-beam LP3
Figure 5.7 Load versus post-tensioning force relationships for box-beam LP3
Figure 5.8  Load versus strain relationships for box-beam LP3
Figure 5.9 Load versus deflection relationships for box-beam DP1
Figure 5.10  Load versus post-tensioning force relationship for box-beam DP1
Figure 5.11  Load versus strain in top flange of box-beam DP1
Figure 5.12  Load versus deflection relationships for box-beam DN2
Figure 5.13 Load versus post-tensioning force relationships for box-beam DN2
Figure 5.14  Load versus strain in top flange at midspan of box-beam DN2
Figure 5.15. Effect of level of pretensioning forces on the load versus deflection response of box-beam, upl = 0.7

Load (kips)

Deflection (in.)

$\rho = 0.67$, 83 kips

$\rho = 0.69$, 87 kips

$\rho = 0.95$, 86 kips

$\rho = 1.5$, 47 kips

$\rho_b = 0.56$, 77 kips

$\rho_b = 0.67$, 83 kips

$\rho_b = 0.69$, 87 kips

$\rho_b = 0.95$, 86 kips

$\rho_b = 1.5$, 47 kips

$\rho_b = 0.56$, 77 kips

$\rho_b = 0.67$, 83 kips

$\rho_b = 0.69$, 87 kips

$\rho_b = 0.95$, 86 kips

$\rho_b = 1.5$, 47 kips
Figure 5.16  Effect of level of unbonded post-tensioning forces on the load-deflection response of box-beams
Figure 5.17  Cross-section configuration of box-beam

Figure 5.18  Flexural strains in tendons for balanced section
Figure 5.19  Cross-section details of box-beam designed for shear
CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Based on the experimental results, the following conclusions can be drawn.

1. The uni-axial stirrup strengths are lower than the strengths of strands in the direction parallel to the fibers. The embedded length, anchorage type, and tail length of stirrup of a specific bend radius and diameter affects the stirrup strength.

2. For a specific radius of bend and fiber type, the stirrup strengths are higher for larger embedded length to effective diameter ratios. The percentage reduction in uni-axial strength of 7.5 mm and 10.5 mm (0.3 and 0.41 in.) diameter CFTC $1 \times 7$ stirrups, and C-bar stirrups due to bend are 30, 65 and 72 percent, respectively. The corresponding reduction in strength of 7.5 mm and 10.5 mm (0.3 and 0.41 in.) diameter CFCC $1 \times 7$ stirrups are 40 and 50 percent, respectively.

3. The effective stirrup stress model in conjunction with uni-axial strength model estimates the shear strength of box-beams with reasonable accuracy. The difference in theoretical and experimental values of the shear strengths of box-beams is less than 2 percent.

4. Bond strength of CFRP bars/tendons depends on their material and surface characteristics rather than the compressive strength of concrete. The average bond strengths for CFRP bars/tendons and steel bars are 1.05 and 2.1 ksi (7.2 and 14.5 MPa), respectively. The DCI bars were observed to have superior bond characteristics to that of MIC C-bars and Leadline tendons.

5. All box-beams tested for shear, failed due to widening of shear cracks followed by rupture of the pre-tensioning tendons due to dowel action. However, the CFRP stirrups and unbonded post-tensioning tendons did not rupture even after the failure of box-beams.

6. The shear strength of beams reinforced with CFRP stirrups was higher than that reinforced with steel stirrups due to higher values of induced effective stresses in the CFRP stirrups. The shear capacity of box-beams M2, M3, T2, T3, S2, and N0 were
257, 267, 226, 291, 223, and 177 kN (57.8, 60.0, 50.9, 65.5, 50.1, and 40 kips), respectively. The maximum strain developed in stirrups of beam M2, M3, T2, T3, and S2 was 23.5%, 17.6%, 25%, 16.6%, and 16.6%, and 95% of the corresponding specified ultimate strain of stirrups, respectively.

7. The energy ratios of box-beams tested for shear and reinforced with CFRP stirrups are lower than that observed for corresponding box-beam reinforced with steel stirrups. Shear failure of box-beams was brittle and sudden and led to the lower energy ratio in comparison to that for under-reinforced box-beams, which failed in flexure. The lower energy ratio of box-beam was due to rupture of bonded pretensioning tendons prior to crushing of concrete.

8. The average value of the measured transfer lengths of box-beams was 337.8 mm (13.3 in.), and the measured transfer lengths varied from those predicted by transfer length equation of Grace (2000a) by about 24 to 51%. The amount of transverse reinforcement has little effect on the measured transfer lengths.

9. The presence of the post-tensioning forces increases the decompression loads and the losses in prestressing forces in the pretensioned tendons by about 68% and 5%, respectively.

10. Box-beam reinforced and prestressed using Leadline tendons exhibited about 14% higher flexural load carrying capacity than the box-beams reinforced and prestressed with DCI tendons.

11. The combination of bonded and unbonded prestressing tendons significantly increases the ultimate moment capacity of box beams. For the same tendon materials, the beam without prestress in unbonded post-tensioning tendons exhibited 20% lower load carrying capacity than the beam prestressed using both the bonded pretensioning and unbonded post-tensioning tendons.

12. Flexural failure of all the box-beams was due to rupture of the bonded prestressing tendons followed by crushing of concrete. Unbonded post-tensioning tendons remained intact without rupture even after the ultimate failure of the beams. However, the post-tensioning force in unbonded post-tensioning tendons was not effective in improving the ductility of box-beams due to inherent under-reinforced box-beam behavior. Furthermore, the presence of unbonded tendons without post-
tensioning forces increased the ductility of under-reinforced box-beams prestressed using only pretensioned tendons by 18%.

13. The comparison of the analytical and experimental results validated the shear and flexural design approaches and the accuracy of the developed special purpose computer program for CFRP prestressed concrete box-beams. The slight difference in the analytical and experimental flexural responses of box-beams was due to applied loading and unloading cycles.

14. For a given level of unbonded post-tensioning forces, the level of pretensioning forces in the bonded tendons significantly affect the overall flexural behavior, ultimate load carrying capacity, and failure modes of the box-beams.

15. The effective pretensioning force level (bpl) of 0.3 in conjunction with post-tensioning force level of (upl) of 0.6, resulted into the maximum ultimate load carrying capacity with crushing of concrete prior to rupture of bonded pretensioning tendons. The pretensioning levels (bpl) greater than or equal to 0.4 resulted into rupture of bonded pretensioning tendons before crushing of concrete.

16. For a fixed value of pretensioning force level and reinforcement ratio, the higher level of unbonded post-tensioning forces results in the higher load carrying capacity of the box-beam provided the unbonded tendons remain intact until the ultimate failure of the box-beam.

17. Box-beams pretensioned and post-tensioned with Leadline™ tendons experienced the highest load carrying capacity and lowest midspan deflection in comparison to beams prestressed with CFRP (DCI) tendons. This is attributed to the higher ultimate strength of Leadline™ tendons in comparison to that of CFRP (DCI) tendons. The ultimate load carrying capacity of box-beams prestressed with bonded pretensioning and unbonded post-tensioning DCI tendons was 18% lower than that of similar beam prestressed with Leadline™ tendons.

6.2 Recommendations

Based on the present study, following recommendations can be made.
1. Uni-axial CFRP stirrup strength models in conjunction with effective stress model could be efficiently used to determine the shear strength contribution of CFRP stirrups. The shear strength contribution of concrete could be reasonably well estimated using established ACI code equations.

2. The use of T-beam tests rather than the pullout tests is recommended to measure the bond strength of CFRP bars because development of tensile stresses in concrete around the bars simulates the real beam situation for bars.

3. Unified design approach (Grace and Singh, 2002) could be effectively used for the design of the box-beam prestressed using bonded pretensioning and unbonded post-tensioning tendons arranged in vertically distributed layers along with non-prestressing tendons with any combination of material characteristics.

4. Further research is required to develop an analytical model to determine the bond strength of CFRP bars. The analytical model should incorporate the material and surface characteristics of the bar in addition to the strength of concrete.

5. Flexural tests on over-reinforced box-beam sections are essential to examine the improvement in the ductility of the beams in comparison to that observed for under-reinforced beams.
REFERENCES

5. ACI 440.1R (2001), “Guide for the Design and Construction of Concrete Reinforced with FRP Bars,” American Concrete Institute, Farmington Hills, MI.
8. ACI 318 (2002), “Building Code Requirement for Structural Concrete (318-02) and Commentary (318R-02),” American Concrete Institute, Farmington Hills, MI.


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A.1 Calculation of Bond Strength

Since bond stress is known to vary along the length of the bar, the average bond strength is represented as the bond strength of the bar and can be evaluated using the following equation.

\[ u = \frac{T}{\pi \times d_b \times l_b} \]  \hspace{1cm} (A.1)

where \( d_b \) is the bar diameter, \( l_b \) is the bond length, and \( T \) is the tensile force acting on the bar. Two methods were used to calculate the value of \( T \); bending moment method and strain method.

A.1.1 Bending moment method

This is the primary method to determine the force \( T \). In this method, the tensile force on the bar is calculated by equating the external and internal moments acting on the beam. Forces acting on the beam are shown in Figure A.1 and the force on the bar is calculated as follows:

\[ T = \frac{P \times a}{d_{arm}} \]  \hspace{1cm} (A.2)

where, \( P \) is the reaction at the support, \( d_{arm} \) is the moment arm of the CFRP bar and \( a \) is the distance from the support to the upper end of the major crack.
Figure A.1  Internal and external forces acting on the T-beam
A.1.2 Strain method

Two strain gages were installed on the bar at the middle portion and readings were recorded. Using the strain at the failure load, the force on the bar was calculated using the following equation.

\[ T = E_f \times \varepsilon \times A_f \]  

(A.3)

where, ‘\( E_f \)’ is the modulus of elasticity, ‘\( \varepsilon \)’ is the measured strain, and ‘\( A_f \)’ is the cross sectional area of the bar.

This method was used to verify the values of ‘\( T \)’ calculated from the bending moment method.

A.2 Calculation of Effective Losses in Pretensioning Forces

In this section, effective prestress losses have been computed using measured decompression loads for beams DP1, DN2, and LP3.

(a) DCI Tendons (Beam DP1)

Total initial pretensioning force = 147.5 kips (Table 3.8)
Total prestressing force = 271 kips (Table 4.7)
Total applied post-tensioning force = 271-147.5
\[ = 123.5 \text{ kips} \]
Decompression load = 31.5 kips (see Figure 4.55)

Moment due to decompression load = \( 0.5 \times 31.5 \times \frac{80}{12} \)
\[ = 105 \text{ kip-ft} \]
Dead load of beam = 0.375 kip/ft

Dead load moment = \( \frac{0.375 \times 15^2}{8} \)
\[ = 10.55 \text{ kip-ft} \]
Stress at the extreme tension fiber due to decompression load, $\sigma_o = \frac{105 \times 12 \times 6}{5344}$

$= 1.41 \text{ ksi}$

The stress due to decompression load at the extreme tension fiber should be equal to the stress at the extreme tension fiber due to combined action of dead load moment, effective pretensioning force, and post-tensioning force.

Thus,

$$\sigma_o = - \frac{M_D}{S_b} + \frac{F_{pre} \times e_b}{A} + \frac{F_{post} \times e_u}{S_b}$$  \hspace{1cm} (A.4)

From Eq. A.4,

$$1.41 = - \frac{10.55 \times 12 \times 6}{5344} + \frac{F_{pre}}{360} + \frac{F_{pre} \times 4 \times 6}{5344} + \frac{123.5 \times 12}{360} + \frac{123.5 \times 6 \times 12}{5344}$$  \hspace{1cm} (A.5)

$$= - 0.14 + 0.0073 F_{pre} + 0.62$$

From Eq. A.5,

Total effective pretensioning force, $F_{pre} = 127.4$ kips

Percent loss in pretensioning force = \(\frac{(147.5 - 127.4)}{147.5} \times 100\)

$$= 13.6\%$$

Thus, percent loss in the prestressing forces of DCI pretensioning tendons of beam DP1 = 13.6%

(b) DCI Tendons (Beam DN2)

Total initial pre-tensioning force = 147 kips (Table 4.7)

Total post-tensioning force = 0.0 kips (Note: No post-tensioning force was applied for this beam).

Decompression load = 18.8 kips (see Figure 4.55)

Moment due to decompression load = $0.5 \times 18.8 \times \frac{80}{12}$

$$= 62.67 \text{ kip-ft}$$
Stress at the extreme tension fiber due to decompression load = \(\frac{62.67 \times 6 \times 12}{5344}\)
\[= 0.844 \text{ ksi}\]

The dead load stress = 0.14 ksi (same as for beam DP1)

Total stress due to effective pretensioning force = 0.0073 \(F_{\text{pre}}\) (computed as for Beam DP1 using Eq. A.5)

Stress at the extreme tension fiber due to post-tensioning force, \(F_{\text{post}} = 0.0\)

From Eq. A.4,

\[F_{\text{pre}} = 134.8 \text{ kips}\]

Percent loss in pretensioning force = \(\frac{(147 - 134.8)}{147} \times 100\)
\[= 8.3 \%\]

Thus, the loss in prestressing forces of DCI pretensioning tendons of beam DN2 is equal to 8.3%.

(c) **Leadline Tendons (Beam LP3)**

Total prestressing force = 270 kips (Table 4.7)

Total initial pretensioning force = 147 kips (Table 4.3)

Total applied post-tensioning force = 270 - 147
\[= 123 \text{ kips}\]

Decompression load = 32 kips (see Figure 4.55)

Moment due to decompression load = \(0.5 \times 32 \times \frac{80}{12}\)
\[= 106.7 \text{ kip-ft}\]

Stress at the extreme tension fiber due to decompression load = \(\frac{106.7 \times 12 \times 6}{5344}\)
\[= 1.438 \text{ ksi}\]

Stress at the extreme tension fiber due to dead load = 0.14 ksi (same as for Beam DP1)

Stress at the extreme tension fiber due to effective pretensioning force, \(F_{\text{pre}} = 0.0073 \times F_{\text{pre}}\) (computed as in Eq. A.5 for beam DP1).

Total stress at the extreme tension fiber due post-tensioning force = 0.618 ksi
Total stress at the extreme tension fiber due to effective pretenioning force, $F_{pre}$

$$= 0.0073 F_{pre} \text{ (computes as for Beam DP1 using Eq. A.5)}$$

From Eq. A.4,

$$F_{pre} = 131.5 \text{ kips}$$

Percent loss in the pretensioning forces of Leadline pretensioning tendons of beam LP3

$$= \frac{147 - 131.5}{147} \times 100$$

$$= 10.5 \%$$

Thus, the loss in the prestressing forces of Leadline pretensioning tendons of beam LP3 is equal to 10.5%.

A.3 Notations

The following symbols are used in this report.

$a$ = distance from the support to the upper end of major crack, in.

$A$ = cross-sectional area of concrete, in.$^2$

$A_{fb}$ = cross-sectional area of bottom bonded prestressing tendons in each row, in.$^2$

$A_{fi}$ = cross-sectional area of tension reinforcement of a particular material, in.$^2$

$A_{fn}$ = cross-sectional area of non-prestressing bars in each row

$A_{fnb}$ = cross-sectional area of non-prestressing bars at bottom of flange, in.$^2$

$A_{fnt}$ = cross-sectional area of non-prestressing bars at top of flange, in.$^2$

$A_{fu}$ = total cross-sectional area of unbonded post-tensioning tendons, in.$^2$

$A_{fv}$ = cross-sectional area of main stirrups, in.$^2$
\( A_{pb} \) = total cross-sectional area of bonded pretensioning tendons, in.\(^2\)

\( A_v \) = minimum area of shear reinforcement, in.\(^2\)

\( b \) = width of compression face of member, in.

\( b_w \) = equivalent web width of beam cross-section, in.

\( c_2 \) = distance from the centroid of the concrete section to the bottom face, in.

\( C \) = resultant compression force in concrete, kips

\( C_R \) = resultant compression force, kips

\( c.g.c \) = axis passing through the centroid of concrete cross-section of the DT-beam

\( c.g.p \) = axis passing through the center of gravity of the resultant pretensioned force

\( d \) = distance of center of gravity of the resultant tension force from the extreme compression fiber, in.

\( \bar{d} \) = distance of center of gravity of the resultant compression force from the extreme compression fiber, in.

\( d_b \) = diameter of tendon/bar, in.

\( d_l \) = distance of centroid of non-prestressing bars at bottom of flange from the extreme compression fiber, in.

\( d_m \) = distance of centroid of bottom bonded prestressing tendons (m\(^{th}\) row) from the extreme compression fiber, in.
\[ d_t = \text{distance of centroid of non-prestressing bars at top of flange from the extreme compression fiber, in.} \]

\[ d_u = \text{distance of centroid of unbonded post-tensioning tendons from the extreme compression fiber, in.} \]

\[ e_b = \text{eccentricity of resultant pretensioning force from the centroid of the precast concrete cross-section, in.} \]

\[ e_u = \text{eccentricity of unbonded post-tensioning tendons from centroid of the concrete cross-section, in.} \]

\[ E_c = \text{modulus of elasticity of concrete, psi} \]

\[ E_f = \text{modulus of elasticity of CFRP pretensioning tendons, ksi} \]

\[ E_{fn} = \text{modulus of elasticity of non-prestressing bars, ksi} \]

\[ E_{fp} = \text{modulus of elasticity of unbonded post-tensioning tendons, ksi} \]

\[ f_{2p} = \text{concrete compressive stress at the bottom face resulting from the axial and bending effects of eccentric prestress force, ksi} \]

\[ f_c = \text{stress in the concrete at extreme compression fiber in significantly under-reinforced beam, ksi} \]

\[ f'_c = \text{specified compressive strength of concrete, ksi} \]

\[ f_{cc} = \text{compressive stress at the centroid of the concrete due to effective prestress force, ksi} \]

\[ f_{ct} = \text{stress in the concrete at extreme compression fiber in over-reinforced beam, ksi} \]
\[ f_{fb} = \text{strength of stirrup at bend, ksi} \]

\[ f_{fu} = \text{specified tensile strength of bonded prestressing tendons, ksi} \]

\[ f_{fun} = \text{specified tensile strength of non-prestressing bars, ksi} \]

\[ f_{fup} = \text{specified tensile strength of unbonded post-tensioning tendons, ksi} \]

\[ f_{fv} = \text{effective stress in the stirrups, ksi} \]

\[ f_o = \text{flexural stress in the concrete at the bottom face of the beam due to self-weight (without load factor), ksi} \]

\[ f_{pbj} = \text{total stress in bonded prestressing tendons of an individual row, ksi} \]

\[ f_{pbm} = \text{total stress in bottom prestressing tendons (mth row), ksi} \]

\[ f_{pbmi} = \text{initial effective prestress in bottom bonded prestressing tendons (mth row), ksi} \]

\[ f_{pnb} = \text{total stress in non-prestressing rods provided in bottom layer of flange, ksi} \]

\[ f_{pni} = \text{total stress in non-prestressing rods of an individual layer along the depth of box-beam cross-section, ksi} \]

\[ f_{pnt} = \text{total stress in non-prestressing bars provided in top layer of flange, ksi} \]

\[ f_{pu} = \text{total stress in unbonded post-tensioning tendons, ksi} \]

\[ f_r = \text{modulus of rupture of concrete, psi} \]

\[ f_u = \text{ultimate strength of stirrup bars, ksi} \]
\( F_{pb} \) = resultant tensile force in bonded pretensioning tendons of an individual row, kips

\( F_{pbm} \) = resultant tensile force in bonded bottom \((m^{th} \text{ row})\) prestressing tendons, kips

\( F_{pi} \) = total initial effective pretensioning and post-tensioning forces, kips

\( F_{pnb} \) = resultant compression force in non-prestressing bars at bottom of flange, kips

\( F_{pnk} \) = resultant tensile force in non-prestressing bars of bottom row \((k^{th} \text{ row})\) in webs, kips

\( F_{pnt} \) = resultant compression force in non-prestressing bars at top of flange, kips

\( F_{post} \) = resultant effective post-tensioning force in unbonded tendons, kips

\( F_{pre} \) = resultant effective pretensioning force in bonded tendons, kips

\( F_{pu} \) = resultant tensile force in unbonded post-tensioning tendons, kips

\( F_R \) = resultant of the tensile forces in bonded and unbonded tendons, kips

\( h \) = total depth of box-beam cross-section, in.

\( h_j \) = distance of centroid of non-prestressing rods of an individual row along the depth of box-beam cross-section, in.

\( h_k \) = distance of centroid of bottom non-prestressing bars \((k^{th} \text{ row})\), in.

\( I \) = moment of inertia of concrete cross-section, in.\(^4\)
\( k_u \) = neutral axis depth coefficient

\( L \) = effective span of the beam, ft

\( L_u \) = horizontal distance between ends of the post-tensioning strands, ft

\( m \) = number of rows of bonded tendons

\( M \) = applied maximum moment due to service loads, kip-ft

\( M_{cr} \) = cracking moment capacity of section, kip-ft

\( M_D \) = maximum bending moment due to dead load, kip-ft

\( M_L \) = maximum bending moment due to live load, kip-ft

\( M_{\text{max}} \) = maximum bending moment at the section under consideration, kip-ft

\( M_n \) = nominal moment capacity of section, kip-ft

\( M_{\text{required}} \) = required moment capacity of the section, kip-ft

\( M_u \) = design moment capacity of section, kip-ft

\( n \) = depth to the neutral axis from the extreme compression fiber

\( \text{N.A.} \) = neutral axis of the box-beam section

\( p \) = number of materials used for tension reinforcement

\( P_{cr} \) = midspan load causing cracking of DT-beam
q = total number of layers of bonded prestressing tendons and non-prestressing bars

r_b = radius of stirrup bend, in.

s = center-to-center spacing of stirrups, in.

S_b = section modulus corresponding to the bottom extreme fiber of composite section, in.³

V_c = shear strength contribution of concrete, kips

V_{ci} = shear strength contribution of concrete based on inclined flexure-shear cracking, kips

V_{cw} = shear strength contribution of concrete based on web-shear cracking, kips

V_d = design shear strength of the beam, kips

V_D = shear force due to superimposed dead load, kips

V_i = shear force at the section under consideration, kips

V_L = shear force due to superimposed live load, kips

V_n = nominal shear strength of the beam, kips

V_o = shear force due to self-weight of the beam (without load factor), kips

V_p = vertical component of the prestress force that normally acts in the opposite sense to the load induced shear, kips

V_s = shear strength contribution of stirrup, kips
\[ V_{sd2} \] = shear strength contribution of stirrups provided at center-to-center spacing of \( d/2 \)

\[ V_{sd3} \] = shear strength contribution of stirrups provided at center-to-center spacing of \( d/3 \)

\[ W \] = total midspan load, kips

\[ W_D \] = self weight of beam/unit length, kips/ft

\[ y_{tc} \] = distance of centroid of box-beam cross-section from the extreme compression fiber, in.

\[ \alpha, \beta \] = Whitney’s rectangular stress block factors

\[ \alpha_1, \beta_1 \] = stress block factors for rectangular section equivalent to parabolic stress-strain relationship for concrete

\[ \alpha_i \] = ratio of specified tensile strength of a particular reinforcement to the specified tensile strength of bonded pretensioning tendons

\[ \delta \] = maximum midspan deflection of box-beam under service loads, in.

\[ \delta_a \] = maximum midspan deflection of box-beam due to applied load, in.

\[ \delta_d \] = maximum midspan deflection of box-beam due to dead load, in.

\[ \delta_p \] = maximum midspan deflection of the beam due to prestressing forces, in.

\[ \Delta \varepsilon_{pu} \] = flexural strain in unbonded post-tensioning tendons

\[ \varepsilon_b \] = strain at bottom of beam at specific load stage
\[ \varepsilon_{cr} = \text{strain at extreme tension fiber at first cracking of beam} \]

\[ \varepsilon_{cu} = \text{ultimate compression strain in concrete} \]

\[ \varepsilon_{fbj} = \text{flexural strain in bonded pretensioning tendons of an individual row} \]

\[ \varepsilon_{fbm} = \text{flexural strain in bonded prestressing tendons of bottom row (m}\text{th row}) \]

\[ \varepsilon_{fu} = \text{ultimate tensile strain capacity of bonded prestressing tendons} \]

\[ \varepsilon_{fun} = \text{ultimate tensile strain capacity of non-prestressing bars} \]

\[ \varepsilon_{fup} = \text{ultimate tensile strain capacity of unbonded post-tensioning tendons} \]

\[ \varepsilon_{ptj} = \text{total strain in bonded pretensioning tendons of an individual row} \]

\[ \varepsilon_{ptji} = \text{initial strain in bonded pretensioning tendons of an individual row} \]

\[ \varepsilon_{pbm} = \text{total strain in bonded pretensioning tendons of m}\text{th row} \]

\[ \varepsilon_{pbi} = \text{initial strain in bonded prestressing tendons of m}\text{th row} \]

\[ \varepsilon_{pub} = \text{total strain in non-prestressing bars provided in the top level of flange} \]

\[ \varepsilon_{pnj} = \text{total strain in non-prestressing bars of an individual row} \]

\[ \varepsilon_{pk} = \text{total strain in non-prestressing bars of bottom row (k}\text{th row}) \]

\[ \varepsilon_{put} = \text{total strain in non-prestressing bars provided in the bottom level of flange} \]
\[ \varepsilon_{pu} = \text{total strain in unbonded post-tensioning tendons} \]

\[ \varepsilon_{pui} = \text{initial strain in unbonded post-tensioning tendons} \]

\[ \varepsilon_t = \text{strain at the top of the beam at specific load stage} \]

\[ \Omega = \text{bond reduction coefficient for uncracked section} \]

\[ \Omega_c = \text{bond reduction coefficient for cracked section} \]

\[ \Omega_u = \text{bond reduction coefficient at ultimate} \]

\[ \phi = \text{strength reduction factor} \]

\[ \phi_{cr} = \text{curvature of the box-beam at first cracking} \]

\[ \rho = \text{tensile reinforcement ratio} \]

\[ \rho_b = \text{balanced reinforcement ratio} \]

\[ \sigma_{bp} = \text{resultant prestress at the extreme tension fiber of the beam due to effective pretensioning and post-tensioning forces} \]

\[ \sigma_o = \text{stress at extreme tension fiber beam due to decompression load} \]