UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 1: Summary Report

by

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16. Abstract

Repair and strengthening techniques using adhesive bonded carbon fiber reinforced plastic or polymeric (CFRP) laminates (also called sheets, tow sheets, and thin plates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:
Volume 1 – Summary Report
Volume 2 – Literature Review
Volume 3 – Behavior of Beams Strengthened for Bending
Volume 4 – Behavior of Beams Strengthened for Shear
Volume 5 – Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications

Volume 1 (this volume) summarizes the main findings of the project. Since the adhesive-bonded plate repair and strengthening technique applies to plain, reinforced and prestressed concrete structures, as well as steel and timber structures, the experience gained during this project and the technology transfer developed cover a wide range of future applications.

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The opinions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors.

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ABSTRACT

Repair and strengthening techniques using adhesive bonded carbon fiber reinforced plastic or polymeric (CFRP) laminates (also called sheets, tow sheets, and thin plates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

Volume 1 (this volume) summarizes the main findings of the project. Volume 2 contains an extensive literature survey with information on available technologies, contact members, latest state of knowledge, on-going projects and the like. The experimental study comprised four main parts, described respectively in Volumes 3 to 6, namely: 1) tests of RC beams strengthened in bending; 2) tests of RC beams strengthened in shear; 3) tests in bending and shear of strengthened beams under low temperature (-29°C) and high amplitude cyclic loading; and 4) repeated freeze-thaw exposure of strengthened beams followed by their test in bending. In each experimental part, results are analyzed, compared, discussed, and key conclusions drawn. Volume 7 provides technical specifications based on information provided by he manufacturers of the two CFRP systems used and augmented by the experience accrued during the course of this investigation.

Since the adhesive-bonded plate repair and strengthening technique applies to plain, reinforced and prestressed concrete structures, as well as steel and timber structures, the experience gained during this project and the technology transfer developed cover a wide range of future applications.

1. INTRODUCTION-MOTIVATION

1.1 Background

The maintenance of highway superstructures, particularly bridges, consumes a significant portion of available maintenance funds in the US and other countries as well. Thus the development of effective repair and retrofit techniques has become an essential goal of every federal or state agency responsible for maintenance of such civil infrastructures. These techniques are also increasingly needed to repair damage due to normal use and to environmental factors.

The use of non-metallic reinforcements, especially fiber reinforced plastics or polymers (FRP), utilizing high performance fibers such as carbon, glass, aramid (Kevlar), and others, is seen primarily as a means to avoid corrosion problems otherwise encountered in concrete structures when using conventional steel reinforcing bars or prestressing tendons. These advanced composites are likely to play a significant role in future construction applications particularly in the strengthening and rehabilitation of existing bridges.

Among the many possible alternatives considered with FRP reinforcements, the use of adhesively bonded laminates (also called sheets, tow-sheets, plates), primarily made of carbon fiber reinforced plastics (CFRP), form the basis of a new technology that is being increasingly considered for the repair and strengthening of bridges and highway superstructures. In this technique, an FRP laminate is glued (adhesively bonded) directly to the tensile face (extreme fiber) of a concrete beam and functions as additional tensile reinforcement. Numerous applications have already taken place in Japan and Western Europe and the number of applications is growing in the US and Canada.

Compared to bonded steel plates where length and weight are limitations, CFRP plates offer the following advantages: they are thin, lightweight, non-corrosive, and can be virtually delivered in any length. While their unit cost is higher than that of steel plates, their easier handling in the field makes them cost competitive.

The number of potential applications of the "bonded plate" technique for repair or strengthening purposes is staggering. Examples include: 1) repair of bridge beams damaged by the occasional impact of over-sized trucks; 2) retrofit of bridge beams in which part of the reinforcement was lost due to corrosion; 3) strengthening of bridge beams such as needed when permissible truck loads are increased; 4) strengthening of slab bridges; 5) tunnel lining; 6) strengthening of steel or wood beam; 7) extending the life of utility poles by wrapping around FRP sheets; and 8) repair of aging water and sewer pipes.

The study described in this report was carried out under a project titled: *"Repair and Strengthening of Reinforced and Prestressed Concrete Beams using CFRP Glued-on Plates"* funded jointly by the Michigan Department of Transportation (MDOT Contract No. 96-1067BAB) and by the Great Lakes Center of Truck and Transit Research (GLCTTR). It was aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin carbon fiber reinforced plastic or polymeric (CFRP) laminates are adhesively bonded to the surface of concrete beams in order to strengthen them.

1.2 Main Objectives

The primary objectives of the project were:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) bonded laminates for repair and strengthening of reinforced concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To identify key parameters to insure successful performance;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

1.3 Reports

To achieve the above objectives, several tasks were undertaken over a period of 30 months and are described in the following series of reports having the same general title. The subtitle of each report gives an idea of the different parts of the experimental program carried out. The first report is the present one.

1. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 1 Summary Report

2. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 2 Literature Review
   This report contains a literature review and a comprehensive synthesis of the latest state of knowledge on the adhesive bonded FRP laminate technique.

3. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 3 Behavior of Beams Strengthened for Bending

4. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 4 Behavior of Beams Strengthened for Shear

5. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 5 Behavior of Beams under Cyclic Loading at Low Temperature

6. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 6 Behavior of Beams Subjected to Freeze-Thaw Tests

7. Repair and Strengthening of Reinforced Concrete Beams Using Adhesive Bonded CFRP Laminates:
   Vol. 7 Technical Specifications

Since the adhesive-bonded plate repair and strengthening technique applies to plain, reinforced and prestressed concrete structures, as well as steel and timber structures, the experience gained during this project and the technology transfer developed cover a wide range of future applications.

2. LITERATURE REVIEW (VOLUME 2)

2.1 Introduction

Techniques such as external post-tensioning and epoxy-bonded steel plates have been used successfully to increase the strength of girders in existing bridges and buildings. High strength composite plates are used as an extension of the steel plating method, offering the advantages of composites materials such as immunity to corrosion, a low volume to weight ratio, and unlimited delivery length (in sheet form) thus eliminating the need for joints [ACI 440, 1996].

Composite plates usually are epoxy-bonded to the tension flange and/or web of slabs or girders, increasing their strength in bending and/or shear, and their stiffness. Compared to external post-tensioning, this technique eliminate the need for special anchorages.

Fiber Reinforced Plastic or Polymeric (FRP) composites are defined, in a most generic way, as a polymeric matrix that is reinforced with strong stiff fibers. Unidirectional FRP sheets, plates, or laminates are made of Carbon (CFRP), Glass (GFRP) or Aramid (AFRP) fibers bonded together with a polymer matrix (e.g. epoxy, polyester, vinyl ester). A laminate is made by stacking one or a number of thin layers of fibers and matrix and consolidating them into the desired thickness.

Among existing FRP laminates, CFRPs offer the highest potential as a replacement of steel in typical strengthening applications, because of their combine properties of very high strength, high stiffness, outstanding fatigue performance, and little sensitivity to stress-rupture with time.

2.2 Organization of Volume 2

Volume 2 provides a review of existing literature on the repair and strengthening of reinforced and prestressed concrete beams using external glued-on Fiber Reinforced Plastic (FRP) sheets, particularly Carbon Fiber Reinforced Plastic (CFRP) sheets. A particular emphasis is placed at synthesizing the information so as to allow the reader to first comprehend the material, and then make rational decisions about its use. Useful sources of information and contact persons throughout the US are also gathered.

Chapter 2 compares the mechanical properties of different types of FRP sheets, built with different types of fibers such as Carbon, Glass, Aramid, with the properties of steel plates. It also provides a summary of the technical data, as obtained from different suppliers, of the commercial CFRP sheets and of the epoxy necessary to bond the CFRP sheets to the structural element.

Chapter 3 provides information about various procedures of application of CFRP sheets glued-on to the surface of concrete beams. Specific information concerning surface preparation, mixing of adhesives, application of the CFRP sheet to the structure, and additional limitations and safety precautions is presented.

Chapter 4 surveys first a large number of research projects and field applications of adhesive-bonded FRP sheets for repair and strengthening of concrete structures; these are being developed by universities and technical laboratories around the world. Whenever possible, a summary of projects (research or field applications) that were deemed relevant to the current investigation, is presented. Second, an analysis of the structural behavior of concrete beams externally strengthened by CFRP sheets is presented. Flexural and shear behavior are described as well as the different modes of failure reported in the literature. In a third part, different issues related to durability, that is of concern to the current study, are addressed. Finally, a number of field applications of glued-on FRP sheets are presented.

Chapter 5 summarizes the recommendations for the current study based on what was learned from the literature review. An extensive list of references classified by source, is provided in Chapter 6 as described next.

2.3 Sources and Classification of Information on FRP Research

The process of obtaining the information on FRP materials and research projects, involved numerous contacts with different organizations. The organizations dealing with FRP were classified as follows:

- State and federal agencies: DOTs, FHWA, US Army;
- Universities;
- Other research institutions;
- Companies and other commercial sources.

A summary of contacts (name, address, telephone number) was developed for current and future reference and was included in Table 7.1 of Appendix A of Volume 2. For convenience the summary of research and applications was grouped according to research teams or organizations working together. An appropriate reference code was assigned to each group.

2.4 Summary of Main Findings

- Substantial amount of research done on FRP composites demonstrated the feasibility of the utilization of glued-on sheets as a strengthening technique for concrete structures.

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- Among different FRP laminates reviewed, Carbon FRP composites seem the most suitable for civil engineering applications. They possess excellent mechanical and durability properties.

- The best known CFRP systems currently used are produced by Tonen, Mitsubishi and Sika. Tonen and Mitsubishi products have similar characteristics. The Sika product differs in thickness and a smaller choice of adhesives. Tonen *Forca Tow Sheet* is the most versatile product. The wide choice of different adhesives and primers makes it suitable for different conditions of applications.

- So far, no standardized guidelines have been developed for this technique. To utilize a strengthening system requires strict following of the procedures recommended by each commercial product manufacturer.

- The choice of the fiber strengthening system should in all cases be based on consideration of specifics of the application, such as:
  - purpose of strengthening;
  - the design of the structure or element;
  - conditions of application of FRP system (accessibility, temperature, humidity, degree of structural damage, surface preparation procedures etc.);
  - risks involved;
  - stress levels likely to occur in the retrofit system;
  - duration for which the repair/retrofit is being designed for.

- Durability issues should be addressed by a more intensive experimental program to ascertain the feasibility of this technique under particular environmental conditions.

- One of the key issues in successful application of CFRP composites is proper preparation of adhesive surfaces. The greater the level of damage (or contamination) of concrete cover, the more aggressive methods are necessary to remove the contaminated layer.

- Field testing is recommended as the best way to corroborate findings performed in laboratory conditions.

- The high costs of the CFRP materials used in structural rehabilitation is compensated by the cost savings on labor due to ease of application.

- Not all issues have been fully explored. Further tests are necessary in order to identify the influence of different physical, mechanical and structural factors on performance of FRP laminates. In particular durability behavior of FRP composites requires further investigation.

3. BEHAVIOR OF BEAMS STRENGTHENED FOR BENDING (VOLUME 3)

3.1 Experimental Program

The part of the investigation deals with reinforced concrete beams strengthened in bending. The CFRP laminates are bonded to the extreme tensile face of the beam with fibers oriented along the longitudinal axis of the beam. Experimental results are presented, analyzed, compared, and discussed.

The experimental program comprised fourteen reinforced concrete T-beams (Table 1a). The test parameters included two levels of steel reinforcement ratio before strengthening, and up to four strengthening levels. Two commercially available strengthening systems were tested, the Sika CFRP plate system (CarboDur), and the Tonen CFRP sheet system. Other selective parameters investigated included two different concrete covers; two conditions of cover preparation, three different end anchorage systems of the glued-on sheets, and pre-loading pre-yielding of the beam prior to strengthening. Details of the test parameters are given next. Key results are summarized in Table 1b.

3.2 Test Parameters

The test parameters included the existing steel reinforcement ratio before strengthening, and the strengthening level. For each steel reinforcement ratio, a control beam was tested and compared with CFRP bonded beams having different strengthening levels. The steel reinforcement ratios of the control beams were respectively $0.27\rho_{max}$ and $0.54\rho_{max}$, where the maximum reinforcement ratio, ρ_{max}, is defined as per the AASHTO or ACI Code to represent 75% of the balanced ratio. The strengthening level (that corresponds to the number of CFRP sheets) was determined assuming the total reinforcement of steel and CFRP will not exceed the maximum reinforcement ratio allowed for the beam by the AASHTO Code based on the assumption that the CFRP sheet will fail in tension.

Two strengthening systems were tested: 1) the Sika CFRP plate system (CarboDur), and 2) the Tonen CFRP sheet system. The Sika plate was about 1.2 mm thick and quite rigid, while the Tonen sheet was 0.11 mm thick and flexible like wall paper. The Tonen system was used for 12 beams and the Sika system for 2 beams. For Beam No. 8, the glued-on Sika CFRP plate had a width of 40 mm, which is equivalent in tensile strength to 2 layers of Tonen CFRP sheets (Forca Tow sheet). Following testing, Beam No. 8 was in very good shape even after the interfacial shear failure of concrete. There was no spalling of concrete cover even though the reinforcing bars had yielded and the 40 mm wide CFRP plate was completely delaminated. Beam No. 8 was later re-used as Beam No. 8-1, this time with a CFRP plate 100 mm wide to evaluate a different bond width and strengthening levels.

For one selected set of parameters, two different concrete covers and cover conditions were investigated to study the influence of concrete cover on strengthening effect and mode of failure. The normal clear cover of concrete was taken as 50 mm. For one beam a concrete clear cover of 25 mm was used. Another beam was cast with an initial 25 mm clear concrete cover; however an additional 25 mm repair mortar cover was added prior to strengthening to simulate damaged concrete in real beams.

Table 1a Parameters and variables for the bending tests (f'_c = 55.2 MPa).

Beam No.	Test parameter	Reinforcement ratio, ρ	$A_{s(used)}$ mm^2	M_n/M_{max} %	Forca Tow sheet FTS-C1-30	Strengthen-ing ratio[1] % ()[2]	CarboDur strip (1 layer)	Strengthen-ing ratio[1] % ()[2]
1	Steel reinforcement ratio & Strengthening level	$0.27\rho_{max}$	2#10 2#13 A_s=400	29	0	0 (29)		
2					1 layer	12 (41)		
3					2 layers	24 (52)		
4					4 layers	47 (75)		
5		$0.54\rho_{max}$	4#16 A_s=800	57	0	0 (57)		
6					1 layer	12 (68)		
7					2 layers	24 (80)		
8	Different system (Sika)	$0.54\rho_{max}$	4#16 A_s=800	57			width= 40 mm	22 (78)
8-1		$0.54\rho_{max}$	4#16 A_s=800	57			width= 100 mm	60 (113)
9	Repaired concrete over	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
10	Extended end nchorage	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
11	Pre-loading	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
12	Concrete cover depth 25 mm	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	22 (78)		
13	Cleaned surface	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
14	No anchorage	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		

Note: 1: $M_{FRP}/M_{max.}$x100

2: $(M_{As}+M_{FRP})/M_{max.}$x100, $M_{max.} = M_n$ (when $A_s = A_{smax}$), (f'_c = 55.2 MPa, f_y = 455 MPa, A_{smax} = 1490 mm^2)

All the above values are calculated

Table 1b Summary of main results of bending tests.

Beam No.	Test parameter	Reinforcement ratio	No. of CFRP layer	Failure mode[1]	Ultimate load kN	Strengthening ratio[2] % ( )[3]	Ultimate deflection mm
1	Steel reinforcement ratio & strengthening level	$0.27\rho_{max}$	0	Steel yielding	114.8	0	164
2			1	CFRP rupture	135.0	18 (41)	83
3			2	Interface failure	140.4	22 (81)	56
4			4	Interface failure	160.7	40 (160)	49
5		$0.54\rho_{max}$	0	Steel yielding	188.0	0	88
6			1	Interface failure	209.9	12 (21)	77
7			2	Interface failure	222.0	16 (41)	51
8	Different system (Sika)	$0.54\rho_{max}$	40 mm	Interface failure	209.2	11 (39)	46
8-1			100 mm	Interface failure	250.9	33 (100)	41
9	Repaired concrete cover		2	Interface bond failure	208.2	11 (41)	73
10	Extended end anchorage			Interface failure	220.4	17 (41)	59
11	Pre-loading			Inter-laminar failure	226.2	20 (41)	119
12	Concrete cover depth			Interface failure	221.2	18 (37)	69
13	Cleaned surface			Interface failure	230.8	23 (41)	60
14	No anchorage			Interface failure	215.1	14 (41)	57

1: Steel yielding: Compression failure of top concrete long after reinforcement yielding
CFRP rupture: Tensile failure of CFRP sheet
Interface failure: Interfacial shear failure of concrete just above the epoxy adhesive.
Interface bond failure: Interfacial bond failure between the repair mortar and the existing concrete
Inter-laminar failure: Inter-laminar shear failure between glued-on CFRP sheets
2: Actual strengthening ratio compared to control beam (Beam No. 1 or No. 5)
3: Design strengthening ratio compared to control beam based on the assumption of CFRP tensile failure

To evaluate different anchorage systems, three different anchorage conditions were provided for beams using the Tonen system. One beam had extended end anchorage which means that the glued-on CFRP sheets were extended up to about 50 mm from the supports, without adding the U-shaped wrapped-around end anchorage. Another beam had neither a U-shaped wrapped-around end anchorage nor an extended end anchorage. All other beams strengthened with Tonen sheets had, at both ends, a 100 mm wide U-shaped wrapped-around end anchorage perpendicular to the longitudinal CFRP sheets. The beams strengthened with the Sika system did not have a wrapped end anchorage.

One beam was pre-loaded slightly beyond yielding of steel reinforcing bars to investigate the influence of loading history before the application of CFRP plate. The permanent deflection and maximum crack width at unloading in the pre-cracked beam were 25 mm and 0.9 mm, respectively.

For all beams except one, the concrete surface to be glued on was prepared, for better bond, by grinding with a disk grinder according to the recommendations of the supplier of the strengthening system used. For Beam No. 13, the surface of concrete was simply cleaned with a vacuum cleaner and wiped with a clean cloth to remove any dust. The test parameters and main results are described in Tables 1a and 1b.

3.3 Strengthening Level and Strengthening Ratio

For the Tonen thin sheet system, the term "different strengthening level" referred to a different number of CFRP sheets used, such as one, two, or four. For the Sika system (thicker plate laminate), a "different strengthening level" implied different widths of the CFRP plate used.

The strengthening ratio of a beam is defined as the increment of nominal bending resistance induced by strengthening to the nominal bending resistance of the beam assuming it has a reinforcement ratio $\rho = \rho_{max}$, where ρ_{max} is the maximum reinforcement ratio for under-reinforced sections according to the AASHTO or ACI Codes.

3.4 Conclusions

Based on the observation and analysis of the experimental test results the following conclusions were drawn.

1. The strengthening technique using externally bonded CFRP sheets or plates can significantly improve the ultimate loading capacity of reinforced concrete beams; however, their ultimate deflection is reduced. Moreover, the strengthened beams had, after failure or delamination of the CFRP sheets, a minimum loading capacity and ductility which were almost same as those of the control beam.

2. Strengthening with CFRP sheets can inhibit the growth of large cracks by helping distribute a large number of smaller cracks; it also protect the steel reinforcement from ingress of corrosive agents.

3. In general, normally strengthened beams fail by interfacial shear failure (delamination) within the concrete, instead of by tensile failure of the CFRP sheet or plate.

4. In normally strengthened beams, the increment in ultimate load obtained by strengthening was almost proportional to the strengthening level or number of CFRP sheets. However, this direct relationship should be further confirmed experimentally in beams with strengthening levels and reinforcement ratios higher that those investigated in this study.

5. For a given reinforcement ratio, the ultimate load capacity increases with the strengthening level, or the number of CFRP sheets. However, the steel reinforcement ratio of the reinforced concrete beam to be strengthened, does not seem to have a significant effect on the increment of load at ultimate achieved by strengthening. This implies that the lower the reinforcement ratio, the higher the strengthening effect in terms of percent increase in ultimate load capacity.

6. The ultimate deflection of strengthened beams decreased in comparison to the control beam as the strengthening level increased, thus leading to a lower ductility. This is one of the disadvantages of beams strengthened using CFRP sheets. However, the strengthened beams had, after failure or delamination of the CFRP sheets, a minimum loading capacity and ductility which were almost same as those of the control beam.

7. Beams using the strengthening system with CFRP plate (Sika system) showed the same load versus deflection response as beams using the strengthening system with CFRP sheet (Tonen system), even though the tensile modulus of the CFRP plate was two thirds that of the CFRP sheet. In this investigation where non-trained students were involved, it was found that the system using CFRP plate is easier and more convenient to apply for flexural strengthening than that using the CFRP sheet.

8. The strengthened beam with a smaller concrete cover had slightly higher ultimate load and considerably larger ultimate deflection than the control beam with normal concrete cover.

9. The beam strengthened after having a repaired concrete cover failed by gradual interlaminar debonding at the interface between existing concrete and repair mortar; it led to a ductile behavior, but did not achieve an adequate level of strengthening.

10. Using a U-shaped end anchorage of the CFRP sheet did not help attain higher ultimate loads or deflections, in comparison to having no anchorage. However, extending the sheet up to the supports led to slightly higher ultimate load and deflection. Therefore, the extended end anchorage system is recommended because it is easier to apply.

11. Preparing the concrete surface by grinding prior to the application of CFRP sheets was not more effective than simply vacuum cleaning and wiping the surface. However this conclusion should be further confirmed in real beams with deteriorated concrete surfaces.

12. Pre-loading and pre-cracking a beam beyond reinforcement yielding had no serious influence on the strengthening effect. Therefore, the CFRP glued-on strengthening technique can be applied even to severely damaged beams.

13. The beam that failed by CFRP sheet delamination and was damaged due to severe concrete cover spalling had, upon reloading, the same ultimate load and deflection as the control beam, even though the damage to the cover was severe. This fact can insure some

minimum safety level for beams strengthened using CFRP sheets, should failure by delamination or tension of the sheet occur.

14. Based on the limited number of tests carried out, it seems that the contribution of the shear resistance of concrete to the strength of the interface, linearly increases with the strengthening level. For this conclusion, the interface shear stress of concrete was calculated based on the assumption of equal shear stress along the shear span.

3.5 General Recommendation

Although numerous factors can affect the extent to which a reinforced concrete beam can be strengthened for bending using adhesively bonded CFRP laminates, it seems safe to design for increments of bending strength not exceeding about 20% of the nominal bending resistance of the beam calculated assuming a reinforcement ratio equal to ρ_{max}, where ρ_{max} is the maximum reinforcement ratio defined in the ACI Building Code or the AASHTO Specification. A smaller concrete cover is generally better and there is no need for special anchorages beyond extending the laminate to close to the support or point of zero moment.

6. BEHAVIOR OF BEAMS STRENGTHENED FOR SHEAR (VOLUME 4)

4.1 Experimental Program

For this part of the experimental investigation, the CFRP laminates, when used, are bonded to the web of a beam (symmetrically on each side) with their fibers oriented normally to the longitudinal axis of the beam. The two strengthening systems used for the bending tests (the Sika CFRP plate system (CarboDur), and the Tonen CFRP sheet system) were also used for the shear tests. The Tonen sheet was wrapped in a U-shape around the bottom of the beam and fully covered the two webs. With the Sika plate, strips were cut and bonded symmetrically to the two web faces, along their entire depth; the width and spacing of the strips were determined to provide a strength equivalent to the one layer of Tonen CFRP sheet.

The experimental program comprised three rectangular concrete beams and three T-beams. The test parameters included selectively, two different shear-span ratios (2.5 for the rectangular beams and 3.5 for the T-beams), two levels of longitudinal steel reinforcement ratio before strengthening, and two shear reinforcement levels. The rectangular beams were fabricated without steel stirrups and reinforced for shear using CFRP bonded laminates only. The T-beams were provided with some steel stirrups and strengthened in both bending and shear. The test parameters are summarized in Table 2a and key results are given in Table 2b. The main conclusions are summarized next.

Table 2a Parameters and variables for shear tests.

Beam No.	*Section* type	*Shear reinforcement* (A_v)		*Shear span-to-depth ratio* (a/d)	*Longitudinal Steel reinforcement ratio* (ρ)	*Strength-ening*
		Stirrup	CFRP sheet			
1	102x254 Rect.-section	None	None	2.5	1.41 ρ_{max}	Control
2		None	$A_{v(FRP)}$ Tonen sheet	2.5	1.41 ρ_{max}	Shear
3		None	$A_{v(FRP)}$ Sika plate	2.5	1.41 ρ_{max}	Shear
4	304x304 T-section	0.91 $A_{v\,(max)}$	None	3.5	0.89 ρ_{max}	Control
5		0.91 $A_{v\,(max)}$	$A_{v(FRP)}$ Tonen sheet	3.5	0.89 ρ_{max}	Shear & Bending
6		0.91 $A_{v\,(max)}$	$A_{v(FRP)}$ Sika plate	3.5	0.89 ρ_{max}	Shear & Bending

Note: 1. Actual concrete compressive strength, f'_c = 25.4 MPa. Actual steel yield stress, f_y = 496 MPa and 483 MPa for rectangular and T sections respectively.
2. Maximum shear reinforcement ratio, $A_{v\,(max)}$, is for maximum longitudinal reinforcement ratio, ρ_{max}.

4.2 Conclusions

1. Strengthening for shear using externally bonded CFRP laminates can significantly improve the ultimate loading capacity of reinforced concrete beams having deficiency in shear. Because shear failure is delayed, their ultimate deflection is also significantly increased. In beams insufficiently reinforced for shear, the use of CFRP shear strengthening led to an increase in load capacity of at least 30%.

2. The Tonen CFRP sheet led to a higher shear strengthening effect than the Sika CFRP strips because of its larger bond area and because it was better anchored by wrapping around the web (U-shape). The development of L or Z shaped Sika CFRP plates should improve the strip anchorage and contribute to its increased efficiency.

3. It was generally observed that shear strengthened beams fail by delamination of the CFRP sheet or plate used for shear strengthening, resulting in shear failure of concrete.

4. The tensile stresses generated in the bonded CFRP laminate used for shear are very low compared to their tensile strengths. In this study they were about one twentieth and one seventh the tensile strength of CFRP Tonen sheet and Sika plate, respectively.

Table 2b Summary of main results of the shear tests.

Beam No.	Section type	Shear reinforcement (A_v)		a/d	Longitudinal Reinforce-ment	Failure Mode	Ultimate load, kN	Ultimate deflection, mm	Predicted failure load (bending failure)[1] kN	Predicted failure load (shear failure)[2] kN
		Stirrup	CFRP							
1	Rectangular	None	None	2.5	1.41 ρ_{max}	Shear failure	51.8	3.0	106.75	31.14
2	Rectangular	None	CFRP sheet (Tonen)	2.5	1.41 ρ_{max}	Steel yielding & Delamination	130.5	7.4	106.75	615.16
3	Rectangular	None	CFRP strip (Sika)	2.5	1.41 ρ_{max}	Delamination shear failure	88.1	4.3	106.75	1001
4	T-section	0.91 A_v max (64.5 mm^2)	None	3.5	0.89 ρ_{max}	Steel yielding & shear failure	164.2	15.5	161.75	166.71
5	T-section	0.91 A_v max (64.5 mm^2)	CFRP sheet (Tonen)	3.5	0.89 ρ_{max}	Steel yielding & Delamination	240.2	12.2	257.15	745.75
6	T-section	0.91 A_v max (64.5 mm^2)	CFRP strip (Sika)	3.5	0.89 ρ_{max}	Delamination shear failure	214.5	9.9	301.35	1131.75

Note: 1. See Appendix in Volume 4 for detailed calculations

2. See Appendix in Volume 4 for detailed calculations

5. In this study, shear stresses of concrete at onset of delamination of CFRP laminate used for shear were about $0.06\sqrt{f'_c}$ and $0.10\sqrt{f'_c}$ for Tonen CFRP sheet and Sika CFRP plate, respectively. (These were the average vertical shear stresses calculated from strain gages placed on the CFRP laminate used for shear strengthening).

6. The strains in the steel stirrups and the CFRP sheets or plates used for shear varied linearly with the applied load in the range following shear cracking and before onset of delamination.

7. The stresses in the CFRP sheet or plate used for flexure increased linearly with the load and deflection in the range prior to delamination of CFRP sheet and plates used for shear.

5. BEHAVIOR OF BEAMS UNDER CYCLIC LOADING AT LOW TEMPERATURE (VOLUME 5)

5.1 Introduction - Description

Four reinforced concrete beams strengthened with CFRP sheets were designed, prepared and tested in bending under low temperature conditions (-29°C). Two short beams designed to be shear critical were strengthened for shear with the CFRP Tonen sheet system. They were tested under center point loading. The two other beams had longer spans; they were designed to be bending critical and were strengthened using the CFRP Sika plate system; they were tested under four points loading. In each case one beam was tested monotonically and the other beam was tested in cyclic fatigue. The amplitude of the cyclic load was taken as 10 to 80% of the failure load observed in the monotonic test. The test parameters and key results are given in Tables 3a and 3b respectively. Details are given in Volume 5. In particular the procedure followed to test the specimens under low temperature conditions is described and the design of a controller system developed to maintain the temperature environment is explained. Also a design example is provided.

Table 3a Parameters for low temperature and fatigue loading tests.

Beam Description	*Temperature*	*Type of Loading*
Bending Testing	-29°C	Monotonic to failure
Bending Testing	-29°C	High amplitude cycling (10% to 80% of ultimate bending strength)
Shear Testing	-29°C	Monotonic to failure
Shear Testing	-29°C	High amplitude cycling (10% to 80% of ultimate shear strength)

Table 3.b Summary of main results for the low temperature and fatigue loading tests.

Beam	Parameter	Predicted load (KN)	Peak Load (KN) Range (%)	Max. Shear (KN)	Max. Moment (KN-m)	Type of Failure
1	Shear Mono-tonic	246	206	103	37	Concrete shear crack from load point to supports. Debonding of the CFRP sheet along the major crack.
2	Shear Fatigue	N.A.	20.6-165 10-80%	82.4	29	Total # cycles = 43083 Fatigue failure of the bottom layer of steel. A vertical crack of the entire concrete section and rupture of the CFRP sheet under the load point.
3	Flexure Mono-tonic	93	131	65.7	50	Partial debonding of the CFRP plate. Interfacial delamination of the CFRP plate at the extreme end.
4	Flexure Fatigue	N.A.	13.1-105 10-80% *9.4-75%	52.6	40	# of cycles = 85,000
		N.A.	13.1-118 10-90% *9.4-85%	59.1	45	# of cycles = 61,000 Total # cycles = 146,000
		N.A.	13.1-131 10-100% *9.4-94%	65.7	50	# of cycles = 9,500 Total # cycles = 155,500
		93	Monotonic 139.15	69.6	53	Total # cycles = 155,500 Partial debonding of the CFRP plate. Interfacial delamination of the CFRP plate at the extreme end. Crushing of concrete at the top layer midspan.

* Modified range according to maximum load of the monotonic test of the "fatigue" specimen.

N.A. Not Applicable.

5.2 Conclusions

1. Prior tests (seen Volume 2) showed that the failure mode of RC beams strengthened with CFRP Sika plates and loaded in monotonic bending at normal room temperature was by delamination of the CFRP plate. The limited tests carried out at low temperature (-29°C) and cyclic fatigue loading, suggest that the failure mode remains the same, that is by delamination.

2. The strain data of the cyclic fatigue test in bending showed redistribution of strains (thus stresses) in the CFRP plate with an increasing number of cycles. A more uniform strain pattern was achieved suggesting that slow delamination of the plate occurred during cycling. Higher strains at the end of the plate confirmed the extension of delamination toward that section and subsequent delamination failure at that section.

3. Values of the interfacial shear stress from the strains recorded by the gages showed that the interfacial strength at failure (1.62 and 1.58 MPa) was similar for both the monotonically tested flexure beam and the fatigue tested flexure beam after 155,500 cycles.

4. Failure in the shear beam subjected to monotonic loading at -29°C occurred by shear delamination of the CFRP Tonen sheet along the web surfaces, followed by shear failure of the concrete across the section. The shear delamination seems to have initiated with the propagation of a diagonal shear crack within the concrete that extended from the load point to the support.

5. Failure in the shear beam that was subjected to cyclic fatigue loading at -29°C was initiated by failure of one of the reinforcing bars in the first layer of steel, which was shortly followed by failure of two additional bars. Subsequent analysis suggested that failure of the rebars was by brittle fracture due to the low temperature at which the tests were performed. It was considered that a combination of low temperature and fatigue contributed to this type of failure.

6. Increase in the shear strain obtained with cyclic shear loading from the rosette gage placed at midspan along the vertical axis of the beam suggest that some delamination and cracking were occurring at that section. It is likely that a shear failure would have occurred in a manner similar to the monotonically tested beam, should failure of the reinforcing bar not have occurred.

5.3 Main Conclusion

The main objective of this part of the research was to ascertain that strengthening by CFRP adhesive bonded laminates remains reliable at low temperatures, such as those experienced in the State of Michigan. The limited number of tests carried out in this study suggests that temperatures as low as -29°C do not affect the behavior of the CFRP strengthening system itself, or the interface on the laminate side.

6. BEHAVIOR OF BEAMS SUBJECTED TO FREEZE-THAW CYCLES (VOLUME 6)

6.1 Test Parameters

The experimental program comprised forty-eight reinforced concrete beams. The beams were prismatic with a cross section of 76.2x76.2 mm and a length of 1016 mm. The beams were minimally reinforced to insure safe handling. The specimens were subjected to up to 300 freeze-thaw cycles. For every parameter, three beams were tested in bending at 0, 100, 200 and 300 cycles, under four points bending. Parameters investigated were two different adhesive systems, the Tonen CFRP sheet system (MBrace), and the Sika CFRP system (Sikadur); and a cracking stage where precracking is meant to simulate the cracking conditions in the field prior to strengthening. It was expected that water would enter into the cracks during the thawing cycle and expand with subsequent freezing, resulting in a more critical situation for the performance of the strengthening system. The influence of the presence of cracks was compared with that of specimens without initial cracks. Control specimens (RC beams with no externally bonded CFRP laminate) were also subjected to 0, 100, 200 and 300 freeze-thaw cycles and tested for comparison. The tests parameters are summarized in Table 4a and the main results are given in Table 4b.

Volume 6 provides all the details of this part of the study. In addition, Volume 6 contains a review of the limited literature pertinent to the subject and two appendices, one giving a detailed design example of a reinforced concrete beam strengthened with CFRP laminates, and the second providing a discussion of different approaches to compute bond shear stress at the interface between the CFRP laminate and concrete.

Table 4a Parameters of the freeze-thaw (F.T.) beam tests.

Parameters	*Number of Freeze-thaw cycles*				*Total number of specimens* (48)
	0	100	200	300	
Control beam (Precracked, no CFRP)	3	3	3	3	12
Sika System (Precracked beams)	3	3	3	3	12
Tonen System (Precracked beams)	3	3	3	3	12
Tonen System (not precracked beams)	3	3	3	3	12

6.2 Conclusions

1. For the control specimens no decrease in the moment capacity or shear strength due to the freeze-thaw (F.T.) cycles was observed. However, a decrease in the maximum deflection was observed.

Table 4b Summary of the main results from the freeze-thaw beam tests.

Number of Cycles	Parameter	Peak Load (KN)	Deflection at Failure (mm)	Delamination Length (mm)	Type of Failure
0 cycles	No CFRP (precracked beam)	2.65	7.62	0.00	Yield and fracture of reinforcement
		2.52	7.11	0.00	Yield and fracture of reinforcement
		2.64	8.03	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	19.54	10.85	101.60	Shear-delamination (crack at 30°angle)
		20.49	12.37	279.40	Shear-delamination.
		19.90	11.15	69.85	Shear-delamination.
	Tonen sheet (non-precracked)	16.79	9.50	196.85	Flexure-delamination
		17.81	9.96	165.10	Flexure-delamination
		17.04	9.55	139.70	Flexure-delamination
	Sika sheet (precracked beam)	22.02	8.76	0.00	Shear (crack length = 178 mm)
		21.29	8.10	0.00	Shear
		23.99	9.58	254.00	Shear-delamination
100 cycles	No CFRP (precracked beam)	2.85	7.62	0.00	Yield and fracture of reinforcement
		2.81	7.11	0.00	Yield and fracture of reinforcement
		2.73	9.65	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	16.09	9.65	127.00	Shear-delamination (45°angle)
		15.81	9.91	152.40	Shear-delamination
		15.93	9.65	165.10	Shear-delamination-flexure (50°angle)
	Tonen sheet (non-precracked)	16.46	10.16	152.40	Shear-delamination
		16.46	9.40	177.80	Shear-delamination
		10.98	8.64	139.70	Shear-delamination
	Sika sheet (precracked beam)	19.55	7.11	0.00	Shear (crack length =229 mm)
		20.39	7.87	0.00	Shear (crack length = 229 mm)
		20.18	7.62	0.00	Shear
200 cycles	No CFRP (precracked beam)	2.94	6.86	0.00	Yield and fracture of reinforcement
		2.96	6.86	0.00	Yield and fracture of reinforcement
		2.53	7.62	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	15.40	6.10	139.70	Shear-delamination (40°angle)
		13.34	6.86	0.00	Shear-delamination
		13.78	7.62	215.90	Shear-delamination (60°angle)
	Tonen sheet (non-precracked)	14.06	7.37	139.70	Flexure-delamination
		13.34	7.37	177.80	Shear-delamination (45°angle)
		14.44	7.62	203.20	Flexure-delamination (75°angle)

	Sika sheet (precracked beam)	19.57	7.62	203.20	Delamination-flexure (25°angle)
		19.64	7.37	241.30	Delamination-shear (45°angle)
		19.49	7.62	0.00	Shear (crack length = 203 mm)
300 cycles	No CFRP (precracked beam)	2.94	6.35	0.00	Yield and fracture of reinforcement
		2.49	6.35	0.00	Yield and fracture of reinforcement
		2.75	6.10	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	12.66	6.86	177.80	Flexure-delamination
		12.45	6.35	165.10	Flexure-delamination
		12.13	6.35	190.50	Shear-delamination (40°angle)
	Tonen sheet (non-precracked)	13.22	7.87	152.40	Shear-delamination
		14.52	7.62	177.80	Shear-delamination (35°angle)
		13.74	8.13	0.00	Shear-delamination
	Sika sheet (precracked beam)	21.88	8.64	177.80	Shear-delamination (35° angle)
		21.33	8.38	241.30	Shear-delamination flexure (25°angle)
		21.14	7.62	254.00	Flexure-delamination (30° angle)

2. For specimens strengthened with CFRP sheets an overall decrease in the moment capacity as well as the maximum deflection was observed with an increase in number of freeze-thaw cycles. Precracked beams using the Tonen system presented the higher rate of decrease of moment capacity (38% average for 300 F.T. cycles). Non-precracked beams also strengthened with the Tonen system led to an average decrease of 20% for 300 F.T. cycles.

3. The maximum moment capacity of beams strengthened with the Sika system decreased 13% on average for 200 F.T. cycles and 4% average for 300 F.T. cycles. This variation was attributed to the influence of different type of failure modes.

4. The average deflection at maximum load was very sensitive to the effect of the freeze-thaw cycles and the cracking condition. For the Tonen precracked beams a reduction of 43% in deflection was found after 300 F.T. cycles whereas for the Tonen non-precracked beams the decrease was of 19%. Beams using the Sika system showed a smaller rate of decrease in deflection at maximum load (15% average for 200 F.T. cycles and 7% for 300 F.T. cycles)

5. With the Tonen system, the values of normalized shear stress (v_n) for the same number of freeze-thaw cycles were higher for the flexure-delamination failure than for the shear-delamination failure. It was concluded that shear cracks accelerate the interfacial crack propagation.

6. With the Tonen system, precracking the beam influences the decrease in the average normalized shear stress with the freeze-thaw cycles. For 300 freeze-thaw cycles, precracked beams had a decrease of 39% (compared with the strength at zero F.T. cycles) whereas the decrease for the non precracked beams was 22%. However the normalized shear stress after 300 F.T. cycles remained almost the same: $v = 0.20\sqrt{f'_c}$ for precracked beams, and $v = 0.19\sqrt{f'_c}$ for non-precracked beams. It can be shown that at zero F.T. cycles the cracking condition influences the capacity of the beam, whereas after 300 F.T. cycles, the freezing and thawing effect dominates.

7. For the Sika system, ignoring vertical shear failure, the decrease in the normalized average shear stress at failure load due to the effect of the freeze-thaw cycles seemed to be less significant than for the Tonen system. A decrease of 10% was observed for 300 freeze-thaw cycles, leading to a value of $0.36\sqrt{f'_c}$.

8. For both strengthening systems (Tonen and Sika), the delamination length found in specimens that failed either by shear-delamination or flexure-delamination was located between the end of the CFRP laminate and the maximum bending moment region.

9. The delamination length was quite uniform for both strengthening systems. For the Tonen system, values of delamination length varied between 140 to 180 mm. For the Sika system, the range was even more narrow, 220-250 mm. No influence of either the number of freeze-thaw cycles or the type of failure mode was observed on delamination length.

6.3 Recommendations

1. Freeze-Thaw (F.T.) cycles influence the behavior of reinforced concrete beams with glued-on Carbon Fiber Reinforced Plastic (CFRP) laminates. According to this study, with the Tonen system a maximum decrease of 29% of flexural capacity could be expected after 200 FT and 38% after 300 F.T. cycles. With the Sika system, the maximum decrease observed was of 13% after 200 F.T. cycles. It should be pointed out that the influence of the Freeze-Thaw cycles may also affects the concrete strength, but its effect cannot be easily observed from testing a reinforced concrete beam in bending; it is possible that this dual effect explains the decrease in strength. Unless some additional tests are carried out, as a first design approximation, it is recommended that a reduction of 40% in horizontal shear strength be taken to account for freeze-thaw exposure.

2. The value of the horizontal interfacial shear strength can be taken conservatively as $0.17\sqrt{f'_c}$ for both strengthening systems. Preliminary analyses indicate that this value may be close to 1.70 MPa for Tonen system and 2.64 MPa for the Sika system, considering the effect of the different strengthening level provided by the two systems. Further study of the interface bond behavior is needed in order to refine this value.

3. The minimum value of the development length (or anchorage) of the CFRP laminate should be based on the value of $0.17\sqrt{f'_c}$. This value could be modified by results from further investigations. It is recommended that the bonded length of the CFRP should be as long as possible in order to avoid an interfacial bond failure and to have a more efficient use of the CFRP sheet strength.

4. Since delamination seems to be controlled by the interface bond between the CFRP laminate and the concrete, which is also controlled by the concrete strength, it is strongly recommended to insure very good surface preparation before application of the strengthening system.

6.4 Correlation with Shear Tests

Average interlaminar shear stresses at failure measured in the beams tested for shear strengthening were $0.06\sqrt{f'_c}$ and $0.10\sqrt{f'_c}$ for the Tonen and Sika system, respectively. Interlaminar shear stresses along the tensile face of the beam calculated at failure for the freeze-thaw test beams were significantly higher, exceeding $0.19\sqrt{f'_c}$. Although the reason for this difference needs to be ascertained in future tests, it is suggested that it could arise from scale effects, due to the fact that the freeze-thaw beams were much smaller that the beams for shear strengthening. Since interlaminar shear failure within the concrete is a brittle fracture failure, a fracture mechanics analysis may provide a rational explanation for this observation.

7. TECHNICAL SPECIFICATIONS (VOLUME 7)

7.1 Summary

These specifications provides sections on description of the technology, materials used including pertinent data for commercial CFRP systems, comparison of commercial adhesives, equipment needed, and execution. For the execution section, details related to assessment of existing conditions, surface preparation, mixing of adhesive, application to the structure, comparison between different commercial systems, and safety precautions are also provided. These specifications are derived by the experience gained during the course of this project and information extracted from different commercial and research sources.

In additions two appendices are included. One dealing with special provisions for using the SIKA CARBODUR (CFRP) laminates for structural strengthening, and the other dealing with special provisions for using the MBRACE (CFRP) composite strengthening system. These special provisions were essentially obtained from the respective suppliers of each system. In case of use of a particular CFRP system, the application procedure provided by the system supplier should be followed.

The successful application and use of these technical specifications is sole responsibility of the user and is dependent on the application of sound judgment by qualified professional engineer with a thorough understanding of concrete behavior and structural mechanics.

8. DESIGN GUIDELINES

A number of design parameters were investigated in this project. However, in many cases only one specimen was tested and results with only one specimen may not be definitely conclusive. The design guidelines suggested next are based on the observed results and the experience gained from the project.

1. Strengthening RC beams in bending or shear using adhesively bonded CFRP sheets is a feasible and reliable technology. However, strengthening leads to a reduction in ductility as evidenced by a reduction in deflection at failure.

2. Strengthening should be limited to balance a portion of live load only. Should the strengthening system fail, the structure under repair should be analyzed (or designed) to resist the dead load and the remaining portion of live load.

3. Although numerous factors can affect the extent to which a reinforced concrete beam can be strengthened for bending using adhesively bonded CFRP laminates, it seems safe to design for increments of bending strength not exceeding about 20% of the nominal bending resistance of the beam calculated assuming a reinforcement ratio equal to ρ_{max}, where ρ_{max} is the maximum reinforcement ratio defined in the ACI Building Code or the AASHTO Specification. Also, in order to comply with the code, it is necessary to keep the total reinforcement ratio or reinforcement index below their respective maximum value recommended in the code.

4. A smaller concrete cover is generally better than a larger one.

5. A repaired concrete cover prior to application of the CFRP laminates is likely to fail at the interface between the old concrete substrate and the new cover. However, the failure will be more ductile than with a virgin cover.

6. U-shaped anchorages are not necessary at the ends of the CFRP laminates. Extending the laminate over its development length should be sufficient.

7. In analyzing or designing a strengthened RC. beam, the design criteria should include not only equilibrium, strain compatibility, and stress-strain relations of component materials, but also criteria related to the interlaminar shear resistance of the concrete both in the direction of bending and shear. There is need to determine the resistance of concrete in shear when subjected simultaneously to tension or compression.

8. Because of possible scale effects and the brittle nature of failure in shear, it is recommended to design on the basis of a concrete interlaminar shear strength not exceeding $0.10\sqrt{f'_c}$ where f'_c is in MPa.

9. Fatigue tests at temperature of -29 ^{o}C showed that the CFRP and epoxy adhesive system remain as effective and reliable as at normal temperatures.

10. Freeze-thaw tests affects mostly the interlaminar shear resistance of the concrete substrate leading to failures modes by interlaminar shear similar to those observed for the control beams.

12. For design it is recommended to reduce the code recommended interlaminar shear resistance of concrete by up to 40% when full cycles of freeze-thaw exposure are expected. The value recommended in 8 above meets this recommendation.

13. For design, it is recommended that the development length of the bonded sheet be calculated based on the interlaminar shear resistance of the concrete to which it will be bonded (see 8 above). As a measure of safety, the full strength of the sheet can be used in computing the development length. However, the development length can also be calculated on the basis of the maximum expected tensile load to be resisted by the bonded sheet, as obtained from strain compatibility analysis.

9. CONCLUDING REMARK

The reader is strongly urged to refer to the details of this investigation as described in Volumes 2 to 7 prior to embarking on an important strengthening project dealing with this new technology.

At time of completion of this project, ACI Committee 440, FRP Reinforcement, was completing two documents of interest to this project: one dealing with design issues, and the other dealing with specifications related to the use of the adhesive bonded laminate technique for repair and strengthening of concrete structures. These references are also highly recommended.

UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 2: Literature Review

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16. Abstract

Repair and strengthening techniques using adhesive bonded carbon fiber reinforced plastic or polymeric (CFRP) laminates (also called sheets, tow sheets, and thin plates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:

Volume 1 – Summary Report
Volume 2 – Literature Review
Volume 3 – Behavior of Beams Strengthened for Bending
Volume 4 – Behavior of Beams Strengthened for Shear
Volume 5 – Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications

Volume 2 (this volume) provides a review of existing literature on the repair and strengthening of reinforced concrete beams using external glued-on Fiber Reinforced Plastic (FRP) sheets, particularly Carbon Fiber Reinforced Plastic (CFRP) sheets. A special emphasis is placed at synthesizing the information so as to allow the reader to first comprehend the material, and then make rational decisions about its use. Useful sources of information and contact persons throughout the US are also gathered. First, the mechanical properties of different FRP sheets, made with different types of fibers such as Carbon, Glass, and Aramid, are compared to the properties of steel plates. A summary of the technical data of the commercial CFRP sheets is then presented. Information about the epoxy necessary to glue the CFRP sheets to the structural element is provided as obtained from different suppliers. Different application procedures of CFRP sheets glued-on to the surface of concrete beams are documented. Specific information concerning surface preparation, mixing of adhesives, application of the CFRP sheet to the structure, and additional limitations and safety precautions is presented for each procedure. The results of an extensive survey of a large number of research projects and field applications of glued-on FRP sheets for repair and strengthening of concrete structures in the US are summarized. Whenever possible, a summary of projects (research or field applications) that are deemed relevant to the current investigation, is presented. An analysis of the structural behavior of concrete beams externally strengthened by CFRP sheets is provided Issues related to durability, which is of concern to the current study, are addressed. A summary of the recommendations for the current study based on what was learned from the literature review is presented, and an extensive list of references classified by source is provided.

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PREFACE

This project titled: *"Repair and Strengthening of Reinforced Concrete Beams using CFRP Laminates"* is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of the project are:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) glued-on plates for repair and strengthening of concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

The project consists of 8 tasks over a period of two years (8 trimesters). The following tasks (accomplishments) are planned:

- A report containing a literature review and a comprehensive synthesis of the latest state of knowledge on the glued -on FRP technique (Task 1);
- Laboratory testing and verification of the selected CFRP glued-on technique according to the proposed experimental program: bending (Task 2), shear (Task 3), freeze-thaw (Task 4), temperature and high cyclic amplitude load (Task 5);
- An interim and final report summarizing the experimental results (Task 6). The interim report will cover the bending and freeze-thaw tests;
- A summary of field specifications and "how to" details for implementation in field applications;
- Guidelines for design based on the experience developed from the experimental work (Task 7);
- Field monitoring of application of the technique to one bridge selected by MDOT (Task 8a);
- Bridge testing before and after application of the glued-on plate (Task 8b to be conducted by professor A. Nowak, U of M)

This volume summarizes the literature review as per Task 1.

ABSTRACT

This report provides a review of existing literature on the repair and strengthening of reinforced and prestressed concrete beams using external glued-on Fiber Reinforced Plastic (FRP) sheets, particularly Carbon Fiber Reinforced Plastic (CFRP) sheets. A special emphasis is placed at synthesizing the information so as to allow the reader to first comprehend the material, and then make rational decisions about its use. Useful sources of information and contact persons throughout the US are also gathered.

First , the mechanical properties of different FRP sheets, made with different types of fibers such as Carbon, Glass, and Aramid, are compared to the properties of steel plates. A summary of the technical data of the commercial CFRP sheets is then presented. Information about the epoxy necessary to glue the CFRP sheets to the structural element is provided as obtained from different suppliers.

Different application procedures of CFRP sheets glued-on to the surface of concrete beams are documented. Specific information concerning surface preparation, mixing of adhesives, application of the CFRP sheet to the structure, and additional limitations and safety precautions is presented for each procedure.

The results of an extensive survey of a large number of research projects and field applications of glued-on FRP sheets for repair and strengthening of concrete structures in the US are summarized. Whenever possible, a summary of projects (research or field applications) that are deemed relevant to the current investigation, is presented. An analysis of the structural behavior of concrete beams externally strengthened by CFRP sheets is provided. Flexural and shear behavior are described as well as the different modes of failure reported in the literature. Issues related to durability, which is of concern to the current study, are addressed.

A summary of the recommendations for the current study based on what was learned from the literature review is presented, and an extensive list of references classified by source is provided.

EXECUTIVE SUMMARY

This report provides a review of existing literature on the repair and strengthening of reinforced and prestressed concrete beams using external glued-on Fiber Reinforced Plastic (FRP) sheets, particularly Carbon Fiber Reinforced Plastic (CFRP) sheets. A special emphasis is placed at synthesizing the information so as to allow the reader to first comprehend the material, and then make rational decisions about its use. Useful sources of information and contact persons throughout the US are also gathered.

Based on the literature review carried out, the following conclusions were drawn:

- Substantial amount of research on and field applications of FRP composites demonstrate the feasibility of the utilization of glued-on sheets as a strengthening technique for concrete structures. A

- Among the different FRP laminates reviewed in this study, carbon FRP composites seem the most suitable (at time of this writing) for civil engineering applications. They possess excellent mechanical and durability properties.

- The best known CFRP systems currently used are produced by Tonen, Mitsubishi and Sika. Tonen and Mitsubishi products have similar characteristics. The Sika product differs in thickness and a smaller choice of adhesives. Because they are very thin and because they can be used in different layers, Tonen *Forca Tow Sheest* are a very versatile product. Tonen offers a wide choice of adhesives and primers for different conditions of applications.

- The simple application procedures developed, have made it possible to utilize CFRP sheets in field applications. However, the application of commercial FRP strengthening systems requires strict following of the specified procedures. So far, no standardized guidelines have been developed for this technique. Proper quality of strengthening can be achieved only by following the procedures recommended by the manufacturer of each system.

- The choice of the fiber strengthening system should in all cases be based on consideration of specifics of the application, such as:

    - purpose of strengthening;
    - the design of the structure or element;
    - conditions of application of FRP system (accessibility, temperature, humidity, degree of structural damage, surface preparation procedures etc.);
    - risks involved;

- stress levels likely to occur in the retrofit system;
- duration for which the repair/retrofit is being designed for.

- Durability issues should be addressed by a more intensive experimental program to ascertain the feasibility of this technique under particular environmental conditions.

- One of the key issues in successful application of CFRP composites is proper preparation of adhesive surfaces. The greater the level of damage (or contamination) of concrete cover, the more aggressive methods are necessary to remove the contaminated layer prior to application of the CFRP plate.

- A field test is recommended as the best way to corroborate findings performed in laboratory conditions.

- The high costs of the CFRP materials used in structural rehabilitation is fully compensated by the ease of application and low costs of workmanship.

- Not all issues related to this new technology have been fully explored. Further tests are necessary in order to identify the influence of different physical, mechanical and structural factors on performance of FRP laminates. In particular durability behavior of FRP composites requires further investigation. Information on fatigue performance and long term behavior is also lacking.

CHAPTER 1
INTRODUCTION

Advanced composites are likely to play a significant role in future construction applications particularly in the strengthening and rehabilitation of existing bridges. According to the Federal Highway Administration, more than 40% of highway bridges in the United States are in need of replacement or rehabilitation ("Highway" 1989). Deficiencies of the existing inventory of bridges range, on the one hand, from normal wear and environmental deterioration of structural components to increased traffic loads demands, and on the other hand, from insufficient detailing at the time of the original design to inadequate maintenance and rehabilitation measures. Therefore, rehabilitation alone will not bring theses bridges up to current standards. Strengthening must also be considered [AZ-03].

Techniques such as external post-tensioning and epoxy-bonded steel plates have been used successfully to increase the strength of girders in existing bridges and buildings. High strength composite plates are used as an extension of the steel plating method, offering the advantages of composites materials such as immunity to corrosion, a low volume to weight ratio, and unlimited delivery length (in sheet form) thus eliminating the need for joints [SOA-1].

Composite plates usually are epoxy-bonded to the tension flange of girders, increasing both their strength and stiffness. The advantages of this strengthening technique include ease of application and elimination of the special anchorages otherwise needed in the post-tensioning method [AZ-1].

Fiber Reinforced Plastic (FRP) composites are defined, in a most generic way, as a polymeric matrix that is reinforced with strong stiff fibers. In construction applications the fiber volume fraction is up to 65%, FRP products were first used to reinforced concrete structures in the mid 1950s. Today, we can find FRP bars, cables, 2-D and 3-D grids, sheet materials, plates, etc. These composites are generally formed by extruding continuous fibers (Carbon, Glass, Aramid, etc.) embedded in a resin matrix. Resins used can be thermoset (polyester, vinyl ester, etc.) or thermoplastic (nylon, polyethylene, etc.) [SOA-1].

The main types of fibers used in civil engineering applications are Carbon, Glass, and Aramid. The most common form of FRP (fiber reinforced plastic) composites used in structural applications is called a laminate. Laminates are made by stacking a number of thin layers (laminate) of fibers and matrix and consolidating them into the desired thickness. Unidirectional FRP sheets made of Carbon (CFRP), Glass (GFRP) or Aramid (AFRP) fibers bonded together with a polymer matrix (e.g. epoxy, polyester, vinyl ester) are being used as a substitute for steel plates [SOA-1].

CFRP and GFRP laminates appear at first to be the best (most economical) candidates for the external reinforcement of concrete structures. However, in view of some properties of GFRP composites such as their sensitivity to ultraviolet radiation, their poor fatigue performance, their stress-rupture sensitivity (i.e. dramatic reduction of their tensile strength over time), and their relatively low modulus of elasticity, it is strongly believed that CFRP laminates (particularly carbon/epoxy systems) offer the highest potential as a replacement of steel in typical strengthening applications. CFRP sheets combine the qualities of very high strength and high stiffness with an outstanding fatigue performance as well as light weight for easy handling. Moreover, CFRP sheets are durable under practically every type of environmental attack which may occur in or around concrete structures [XX-21] [EMPA-12].

1.1. Organization of this Report

This report provides a review of existing literature on the repair and strengthening of reinforced and prestressed concrete beams using external glued-on Fiber Reinforced Plastic (FRP) sheets, particularly Carbon Fiber Reinforced Plastic (CFRP) sheets. A particular emphasis is placed at synthesizing the information so as to allow the reader to first comprehend the material, and then make rational decisions about its use. Useful sources of information and contact persons throughout the US are also gathered.

Chapter 2 compares the mechanical properties of different types of FRP sheets, built with different types of fibers such as Carbon, Glass, Aramid, with the properties of steel plates. It also provides a summary of the technical data, as obtained from different suppliers, of the commercial CFRP sheets and of the epoxy necessary to glue the CFRP sheets to the structural element.

Chapter 3 provides information about various procedures of application of CFRP sheets glued-on to the surface of concrete beams. Specific information concerning surface preparation, mixing of adhesives, application of the CFRP sheet to the structure, and additional limitations and safety precautions is presented here.

Chapter 4 surveys first a large number of research projects and field applications of glued-on FRP sheets for repair and strengthening of concrete structures; these are being developed by universities and technical laboratories around the world. Whenever possible, a summary of projects (research or field applications) that are deemed relevant to the current investigation, is presented. Second, an analysis of the structural behavior of concrete beams externally strengthened by CFRP sheets is presented. Flexural and shear behavior are described as well as the different modes of failure reported in the literature. In a third part, different issues related to durability, which is of concern to the current study, are addressed. Finally, a number of field applications of glued-on FRP sheets are presented.

Chapter 5 summarizes the recommendations for the current study based on what was learned from the literature review.

An extensive list of references classified by source, is provided in Chapter 6.

1.2. Sources and Classification of Information on FRP Research

The process of obtaining the information on FRP materials and research projects, involved numerous contacts with different organizations. The organizations dealing with FRP can be classified as follows:

- State and federal agencies: DOTs, FHWA, US Army;
- Universities;
- Other research institutions;
- Companies and other commercial sources.

A summary of contacts has been developed for current and future reference and is included in Table 7.1 of APPENDIX A (section 7.2). For convenience the summary of research and applications is grouped according to research teams or organizations working together. An appropriate reference code is assigned to each group (e.g. [EMPA-1]).

CHAPTER 2
MATERIALS

Different types of FRP materials are used in Civil Engineering applications for concrete structures:

- Reinforcing bars
- Prestressing tendons
- Rigid plates
- Laminates (sheets)
- Grid or meshes

Laminates are relatively a new technology. Thanks to the remarkable advantages of this material it is increasingly used for different types of repair techniques.

2.1. Features of FRP Sheets in Civil Engineering

FRP sheets can be used for the following purposes:

- Serviceability improvements:
  - Decrease in deformation (primarily deflection);
  - Reduction of stresses in steel reinforcement;
  - Crack width reduction.
- Correction of design/construction defects:
  - Insufficient reinforcements;
  - Insufficient structural depth.
- Upgrade of damaged structural parts:
  - Aging of construction materials;
  - Steel reinforcement corrosion;
  - Vehicle impact;
  - Fire impact.
- Change of loading conditions:
  - Increase of traffic volumes on bridges;
  - Increase of live loads in warehouse buildings;
  - Installation of heavy machinery in industrial buildings;
  - Vibrating structures.
- Structural modifications:
  - Removal of walls or columns;
  - Removal of slabs sections for openings.
- Seismic upgrade.
- Change in use of structure.

2.1.1. CFRP Sheets

Carbon Fiber Reinforced Plastic laminates are composite materials built from combination of carbon fibers and epoxy resin matrix. The composite possesses very high strength and elastic modulus in the fiber direction. Its fatigue properties are also outstanding. All the fibers in the laminate form unidirectional structure. The transversal strength of the composite is low. This drawback is not relevant for many strengthening applications.
In general CFRP can be classified according to:

- High-strength fibers with tensile strength up to 7000 MPa.
- High-modulus fibers with E-modulus up to 600 GPa.

The use of a particular type of CFRP depends on the purpose of application. Consideration has to be given as to whether the necessary repair measures relate to stability, fatigue safety or fitness for use. The engineer has to consider the state of stresses in the strengthened zone - compressive, tensile or shear. Accordingly, the proper type of CFRP system has to be selected and applied in the proper direction or in multiple directions.

The unidirectional FRP materials are produced by a number of companies. Every one of these producers offers Carbon Fiber sheets in a variety of grades. At the time of this writing the majority of the available research papers refer to the following three products:

- CarboDur (Sika Corporation),
- Forca Tow Sheet (Tonen Corporation)
- Replark (Mitsubishi).

The properties of these composite sheets are summarized in Table 2.1. The characteristics of the proprietary adhesives are summarized in section 2.2.1.

2.1.2. Comparison between FRP Sheets and Steel Plates

The strengthening technique of bonding steel plates is widespread around the world and is the state of the art. According to U. Meier from the Swiss Federal Laboratories for Materials Testing and Research (EMPA) for applications in which corrosion plays no role and the length of the strengthening component is less than 5 m., steel offers an optimum solution. However, in applications where corrosion, length of strengthening component and handling on construction sites play a dominant role, FRP laminate must be considered [EMPA- 01].

A comparison between CFRP sheets and steel plates in application to concrete structures is presented in Table 2.2. Part of the table is taken from references [SCD-1] and [EMPA-3].

Table 2.1 Data Summary For Commercial FRP

Property \ Product	Sika *CarboDur*	Tonen *Forca Tow Sheet*		Mitsubishi *Replark*
Type / Grades	Type 50 CarboDur Type 80 CarboDur Type 100 CarboDur	FTS-C1-20 FTS-C1-30 FTS-C5-30	FTS-GE-30 FTS-GT-30	Type 20 (MRK-M2-20) Type 30 (MRK-M2-30) Type MM (MRK-M4-20) Type HM (MRK-M6-30)
Type of Fibers	Carbon fibers Toray T300 &T700	High tensile CF High tensile CF High modulus CF	E-Glass Fibers T-Glass Fibers	Standard Modulus CF Standard Modulus CF Medium Modulus CF High Modulus CF
Type of Matrix	epoxy resin matrix	epoxy resin matrix	epoxy resin matrix	epoxy resin matrix
Tensile Strength	2,400 MPa (348 ksi)	3,480 MPa (505 ksi) 3,480 MPa (505 ksi) 2,942 MPa (427 ksi)	1,516 MPa (220 ksi) 2,694 MPa (391 ksi)	3,400 MPa (493 ksi) 3,400 MPa (493 ksi) 2,900 MPa (421 ksi) 1,900 MPa (276 ksi)
Modulus Of Elasticity 10^6 psi = Msi (1000 N/mm^2 = 1 GPa)	.> 150 GPa (21.75 Msi)	230GPa (33 Msi) 230GPa (33 Msi) 373 GPa (54 Msi)	72.6 GPa (10 Msi) 87.1 GPa (12 Msi)	230 Gpa (33.4 Msi) 230 GPa (33.4 Msi) 390 GPa (56.6 Msi) 640 GPa (92.8 Msi)
Ultimate Elongation At Break [%]	1.4	1.5 1.5 0.8	2.1 3.2	N.A.
Fiber Areal Weight g/m^2 (oz/yd^2)	N.A.	200 (5.9) 300 (7.4) 300 (7.4)	300 (7.4) 300 (7.4)	200 (5.9) 300 (7.4) 300 (7.4) 300 (7.4)
Density ρ	1.6 g/cm^3 0.058 λβ/in^3	1.82 g/cm^3	2.55 g/cm^3 2.50 g/cm^3	1.6 g/cm^3
Thickness	1.2 mm	0.11 mm 0.17 mm 0.17 mm	0.118 mm 0.120 mm	0.11 mm 0.17 mm 0.17 mm 0.14 mm
Sheet width cm (inch)	50 mm (2.0") 80 mm (3.15")	50 cm (19.7") 50 cm (19.7") 50 cm (19.7")	50 cm (19.7") 50 cm (19.7")	25 cm (10") all grades 33 cm (13") all grades 50 cm (20") all grades
Proprietary Adhesive	Sikadur 30 - epoxy resin two component adhesive.	FP-NS, FP-NSS, FP-NSW, FP-S, FP-WE7, FP-WE7W, FR-E3P, FR-E3PS, FR-E3PW	FP-NS, FP-NSS, FP-NSW, FP-S, FP-WE7, FP-WE7W, FR-E3P, FR-E3PS, FR-E3PW	Epotherm Primer, Putty and Resin
Lengths Available	Any length	Standard: 100 m.	Standard: 100 m.	Standard: 100 m (328 ft)

Note: the values of properties are given in a sequence corresponding to grade sequence.

N.A. = Information not available

Table 2.2 Comparison of Strengthening Methods in Reinforced Concrete Structures

Criteria \ Material	FRP Laminate	Steel Plates
Own weight	Low. ρ=1.6 g/cm^3	High. ρ=8.0 g/cm^3
Strength of Material	Very high tensile strength in direction of fibers. Low strength in other directions (shear, delamination).	High strength in all directions.
Mechanical behavior	Brittle	Ductile
Fatigue properties	Outstanding.	Adequate
Overall thickness	Very low. Allows application of strips in multiple directions crossing each other.	Low
Length of strips/plates	Unlimited	Limited. Lap joints may be necessary.
Lap joints	Easy	Complex
Type of adhesive	Epoxy Resin	Epoxy Resin
Application techniques	No expensive equipment necessary. Especially convenient in case of overhead strengthening.	Lifting equipment and clamping devices necessary. Particularly inconvenient in case of overhead strengthening.
Handling	Easy. Flexible material. Can be applied on non-planar surfaces.	Difficult. Rigid material. Cannot be applied on non-planar surfaces.
Material costs	High	Low
Installation costs	Low	High
Corrosion	No corrosion. Alkali Resistant	Interface-corrosion
Durability	Depends on durability of adhesive. Protection against UV necessary.	Depends on durability of adhesive

2.1.3. Comparison Among Different Fibers

Fibers, natural or synthetic, are the fundamental constituent of a fiber-reinforced composite. They generally occupy the largest volume and are the major load carrying element of the composite. In this section, properties of typical fibers are presented. The type of fibers is limited to those that have a potential for application in construction [Az-06]

The choice of FRP material for use in a specific reinforcing application is dictated by the set of physical and mechanical properties of the fiber. Carbon Fiber based composites are the best performer in a majority of Civil Engineering applications because they fulfill most of the requirements (see Table 2.3, [EMPA-1]). Others types of fibers such as Glass and Aramid do not seem to satisfy durability requirements and therefore can not be used in cases where exposure to aggressive agents is expected.

Table 2.3 Qualitative Comparison of the Vulnerability of Different Fibers to Various Environments [EMPA-1].

Property \ Fiber	Carbon Fiber (CF)	Glass Fiber (GF)	Aramid Fiber (AF)
Vulnerable to Moisture	No	Yes	No
Vulnerable to Solvents	No	x	x
Vulnerable to Acids	No	Yes	Yes
Vulnerable to Bases	No	Yes	Yes
Vulnerable to UV radiation	x	x	Yes

2.1.4. Comparison Among Different FRP Sheets

Carbon, Aramid, and Glass fiber reinforced plastics are the three types of composite materials that dominate in Civil Engineering applications. The characteristics of a composite are dictated by the properties of the fiber and matrix as well as the method of fabrication involved and the structure of the composite itself.

Table 2.4 [EMPA-1], lists criteria which specifically relate to the use of composite materials as post-strengthening materials for structures. The table is also applicable to prestressed laminates (the FRP laminate is prestressed with initial force prior to be bonded to the reinforced concrete structure). From Table 2.4, it can be concluded that Carbon Fiber Reinforced polymer Composites are the most suitable for post strengthening of concrete structures.

Table 2.4 Comparison of Different Types of Fiber Reinforced Plastic Sheets [EMPA-1]

Property \ Fiber Sheet	Carbon Fiber (CF)	Glass Fiber (GF)	Aramid Fiber (AF)
Tensile Strength	Very good	Very good	Very good
Compressive Strength	Very good	Good	Inadequate
Modulus of Elasticity	Very good	Adequate	Good
Long Term Behavior	Very good	Adequate	Good
Fatigue Behavior	Excellent	Adequate	Good
Bulk Density	Good	Adequate	Excellent

2.2. Adhesives

Adhesives allow to bond structural elements without changing the appearance of the structure. Bond forces are generated by molecular attraction between adhesive and the bonded materials. The adhesive strength depends on the type of molecules and their reciprocal distance. It can be heavily diminished by factors like: dirt, oil , dust and grease. Therefore thorough preparation of surfaces is of utmost importance. Adhesion to exposed aggregates is generally better than to hardened cement paste.

Bonding with adhesive assures evenly distributed stress transmission over the whole contact interface, in contrast to anchorages such as bolted connections which create stress concentrations.

2.2.1. Properties of Commercial Adhesives

There is a number of companies on the market offering adhesives which are potentially suitable for use with FRP fabrics. Most of these adhesives are organic polymers (epoxy resins).

The following characteristics are considered essential for heavy duty structural adhesives:

- strong adhesion to bonded elements;

- strong cohesion;
- little tendency to creep under load;
- good resistance against humidity and alkalinity.

Epoxy resin adhesives possess dense cross links and meet the above requirements very well.

The main producers of FRP fabrics provide proprietary adhesives, which together with the FRP material create the system. The application of a specific type of adhesive depends on working conditions (temperature, moisture). The diversity of adhesives offer by the main producers of the sheets is as follows:

- Tonen Corporation (most diversified)
- Mitsubishi
- Sika Corporation (least diversified)

A comparison of adhesives from the leading producers is presented in Table 2.5 below.

Table 2.5 Comparison of Commercial Adhesives

Property \ Producer	Sika	Tonen									Mitsubishi						
Grade	Sikadur 30	FP-NS	FP-NSS	FP-NSW	FP-S	FP-WE7	FP-WE7W	FR-E3P	FR-E3PS	FR-E3PW	PS301	PS401	PS318	L525	L700S	L700W	L700R
Material	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy
Type	Adhesive Mortar	Primer Standard	Primer Summer	Primer Winter	Primer Penetr..	Primer Summer Damp Surf.	Primer Winter Damp Surf.	Resin Standard	Resin Summer	Resin Winter	Primer Fall/ Spring	Primer Spring/ Fall	Primer All year	Putty All year	Resin Spring/ Fall	Resin Winter	Resin Tunnel
Appl. Temp. °C (°F)	18-30 (65-85)	15-25 (59-77)	25-35 (77-95)	5-15 (41-59)	5-35 (41-95)	15-25 (59-77)	5-15 (41-59)	15-25 (59-77)	25-35 (77-95)	5-15 (41-59)	5-25 (41-77)	20-35 (68-95)	5-35 (41-95)	5-35 (41-95)	15-35 (59-95)	5-15 (41-59)	5-25 (41-77)
Mixing Ratio: by weight		2:1	2:1	2:1	1:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1
by volume	3:1																
Viscosity, cps	N.A.	2,000	2,000	1,300	90	45,000	45,000	20,000	20,000	10,000	500 (MPa S)	500 (MPa S)	17 (MPa S)	Putty	4,000 (MPa S)	3,500 (MPa S)	4,000 (MPa S)
Pot Life, min																	
@30°C (86°F)			40		120				40		25	140	> 120	30	50		
@23°C (73°F)	70										40	240	> 120	50	70	20	20
@20°C (68°F)		40	120	20	120	40	20	40	110	20							
@10°C (50°F)					120		50			50	95		> 120	100	-	70	70
Setting Time, hrs	N.A.																
@30°C (86°F)									7	7	-	4.0	-	3.0	3.5	-	-
@23°C (73°F)											3.5	7.0	2.5	3.5	6.0	3.5	3.5
@20°C (68°F)		10	12	7	3	12	10	10	12	12	-	-	-	-	-	-	-
@10°C (50°F)											6.5	-	4.0	5.0	-	6.0	6.0
Couring Time,	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.		-	-
days															5	-	-
@30°C (86°F)															7	7	7
@23°C (73°F)															-		
@20°C (68°F)															-	14	14
@10°C (50°F)																	
Compressive Strength (14 day) MPa (psi)	> 58.6 (8,500)	N.A.									N.A.						
Shear Strength MPa (psi)	24.8 (3,600)	12.5 MPa									N.A.						
Tensile Strength (7 day), MPa (psi)	24.8 (3,600)	N.A.									N.A.						
Elong. at Break	1%	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.

Property \ Producer	Sika	Tonen									Mitsubishi						
Grade	Sikadur 30	FP-NS	FP-NSS	FP-NSW	FP-S	FP-WE7	FP-WE7W	FR-E3P	FR-E3PS	FR-E3PW	PS301	PS401	PS318	L525	L700S	L700W	L700R
Bond Strength: - Concrete to conc. - Concrete to steel	21.3 MPA 17.9 MPA	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Shelf Life	2 years in unopened packaging	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Storage Conditions °C (°F)	4-35°C (40-95°F)	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Color Main Agent Hardener	Light gray	N.A.									Pale yellow Brown		Transp. Brown	White Black	Green and thixotropic liquid Brown		

N.A. = Information not available

2.2.2. Research on Adhesives

The mechanical properties of the adhesive used for this particular type of repair system will influence the overall behavior of the strengthened structure, particularly, the strength and durability of the interface layer. Most of the experimental work so far undertaken confirm, that the commercial systems present a failure in the concrete cover rather than in the adhesive interface. This proves that the adhesive posses a sufficient strength. Durability of the interface constitute a major concern and requires further investigation.

Saadatmanesh and Ehsani [AZ-3, AZ-5] studied the behavior of four different types of epoxies, used to glue GFRP sheets for strengthening of concrete beams. Five beams were tested to find the static flexural strength. Four beams were strengthened with GFRP sheets using different epoxies, the fifth was not strengthened and used as a control specimen. Two modes of failure of strengthened beams were recognized: failure of the plate (yielding of steel plate or rupture of fiber composite plate and crushing of concrete in compression) and shear failure of the adhesive concrete layer. It was also found that the maximum achievable force in the plate at failure will be equal to the resultant of the limiting shear stresses. It was concluded that selection of a suitable epoxy is very important in the success of this strengthening technique. The epoxy should have sufficient stiffness and strength to transfer the shear force between the composite plate and concrete. It should also be tough enough to prevent brittle bond failure as a result of cracking of concrete. Rubber toughened epoxies were found to be particularly suitable for this application.

2.3. Comparison of Different CFRP Systems

A comparison of the properties and applicability of different strengthening systems is presented in Tables 2.6 and 2.7. Tonen Forca Tow sheet, Mitsubishi Replark and Sika CarboDur are the three type of commercial systems considered. Mechanical properties and durability data are presented. The tables were developed following analysis of technical data provided by the suppliers.

Table 2.6 Comparison of Properties of Different Strengthening Systems

Supplier		Tonen			Mitsuibishi				Sika
System		Forca Tow Sheet			Replark				CarboDur
CFRP Sheet	Type	FTS-C1-20	FTS-C1-30	FTS-C5-30	MRK-M2-20	MRK-M2-30	MRK-M4-30	MRK-M6-30	CarboDur Strip
	Tensile Strength (MPa)	3,480	3,480	2,942	3,400	3,400	2,900	1,900	2,400 10^3
	Tensile Modulus (GPa)	230	230	373	230	230	390	640	1050
	Thickness[a] (mm)	0.11	0.17	0.17	0.118	0.17	0.17	0.14	1.2
	Ultimate Elongation (%)	1.5	1.5	0.8	N.A.	N.A.	N.A.	N.A.	1.4
	Width	50 cm			25 cm, 33 cm, 50 cm				5 cm, 8 cm cm2",3.2",4"
	Length	100 m Standard			100 m standard				Up to 250 m
Epoxy	Type	FR-E3P (Resin)	FR-E3PS (Resin)	FR-E3PW (Resin)	L700S		L700W	L700R	Sikadur 30
	Application Temperature(°C)	Standard (15°-25°)	Summer (25°-35°)	Winter (4°-15°)	Spring to Fall (15°-35°)		Winter (4°-15°)	Tunnel (5°-25°)	≥39°
	Tensile Strength	N.A.	N.A.	N.A.	N.A.		N.A.	N.A.	24.8 MPa
	Elastic Modulus	N.A.	N.A.	N.A.	N.A.		N.A.	N.A.	4.48 MPa
	Elongation	N.A.	N.A.	N.A.	N.A.		N.A.	N.A.	1%
	Shear Strength	N.A.	N.A.	N.A.	N.A.		N.A.	N.A.	24.8 MPa
	Shrinkage	N.A.	N.A.	N.A.	N.A.		N.A.	N.A.	0.0004
	Pot Life (min.)	40	110	20	70		20	20	70
	Viscosity (cps)	20,000	20,000	10,000	4,000 MPa S		3,500 MPa S	4,000 MPa	N.A.
Primer	Type	Standard, Summer, Winter, Penetrative, Summer Damp Surface, Winter Damp Surface			Fall to Spring, Spring to Fall, All year				No Primer

a: Total cross sectional area of fibers per inch.
N.A. = Information not available

Table 2.7 Comparison of Applicability of Different Strengthening Systems

System / Category	Tonen *Forca Tow Sheet*	Mitsuibishi *Replark*	Sika *CarboDur*
CFRP Sheet	- Easy control of strengthening level with layer -Thin thickness:0.0065in.(0.17mm) (3.3 kips/in.) > For bending strengthening . Slab, box beam: 1-2 layers . I or T beams: 3 to 5 layers > For shear strengthening . Easy wrapping of beam . Sufficient anchorage for shear	Same as Tonen Forca Tow Sheet	- Control of strengthening with strip - Thick thickness: 0.047in (1.19mm) (16.4 kips/in.) > For bending strengthening . Slab, box beam: several strips (only 1 layer) . I or T beams: only 1 layer . Require higher shear interface > For shear strengthening. . Impossibility of wrapping . Insufficient anchorage
Epoxy & Primer	Wide selection of resin systems (4 epoxies & 6 primers) suitable for various concrete surfaces and climate conditions	- 2 epoxies and 3 primers for different climate conditions	- Wide coverage of applicability of the resin,Sikadur-30 (> 39°F) - Tolerant of moisture before, during, and after cure - No primer is needed
Field Applications	- 120 field applications in Japan - Foulk Road Bridge (box beam) , Delaware	-16 field applications - Bridge girder, Florida	- Applications in Europe - Ibach bridge (box beam), Switzerland
Research Programs	- University of Delaware (with Delaware DOT) - West Virginia University - Federal Highway Administration - University of California (with US Navy Engineering Service Center) - Caltrans	- Caltrans - Federal Highway Administration in McLean and Florida - Florida DOT - Georgia Institute of Technology	- EMPA (Federal Materials Testing and Research Center) , Switzerland since 1984

CHAPTER 3
APPLICATION PROCEDURES

This chapter presents the synthesis and the comparison of the application procedures based on the information extracted from different commercial and research sources. In case of use of a particular FRP system, the application procedure provided by the material supplier should be followed.

3.1. Assessment of Existing Conditions

The determination of strengthening technique should be supported by thorough examination of existing structural and environmental conditions such as:

- Conditions of the member;
- Concrete quality;
- Reinforcement configuration and location;
- Member geometry;
- Load conditions;
- Environmental exposition (Road salt, UV light, freeze-thaw).

3.2. Surface Preparation

The bond strength between the adhesive and the materials depends on the type of molecules and their reciprocal distance. Dirt, dust, oil and grease heavily impair the reciprocal attraction. Most engineers tend to focus on achievement of the prescribed initial bond strength. However, bond properties are dictated by environmental stability of the adherent-adhesive interface. This is in particular true in civil engineering applications, where degree of control is limited, and the vagaries of nature such as extreme temperature, moisture, sustained and cyclic loading are very intense.

Therefore, thorough preparation of the surfaces of the parts bonded together is of great importance. The FRP can be applied on sound and clean surfaces. The suppliers of FRP prescribe the necessary actions to achieve suitable surface conditions. The following represents the synthesis of the surface preparation recommendations taken from commercial information and also the research sources. Before application of a particular system, one should refer to the appropriate commercial specifications. All possible bonding surfaces involved in strengthening system (concrete, FRP, steel) should be prepared.

3.2.1. Concrete Surfaces

- In order to provide open roughened texture the application of the following surface preparation techniques can be necessary:
  - Sandblasting (high pressure sand abrading);
  - High-pressure water washing;
  - Bush-hammering;
  - Disk abrading (grinding).
- Surface must be free of standing water. Some types of adhesives require dry surface.
- Watch for the formation of condensation (dew point).
- Removal of dust (vacuum cleaning).
- Removal of grease, curing compounds, impregnation, waxes, foreign particles, disintegrated materials, paints, plasters, wall papers and other bond inhibiting materials.
- Adhesion to exposed aggregates is generally better than to the hardened cement paste. The laitance must therefore be removed and the aggregate must be exposed as gently as possible.
- Filling of gaps, cavities and uneven portions of structure with appropriate repair mortar specified by the system producer. According to Sika AG, an epoxy repair mortar should be apply ([SCD-1]).
- The surface to be coated must be even (on a length of 2 meters, unevenness may not exceed 10 mm [SCD-1]). Steps and formwork marks should not be greater than 0.5mm.
- At the direction of the engineer, the adhesive strength of the concrete surface should be verified after preparation by random pull-off testing (ACI 503R). This testing should prove a minimum tensile strength of the adhesive interface specified by the system supplier.

The comparison of different surface preparation techniques depending on the level of contamination and deterioration and the depth of removal of concrete cover is presented in the Table 3.1 below (based on Sika materials.) The bigger the contamination and the thickness of removed material the more aggressive is the method that have to be applied.

Table 3.1 Methods of Concrete Surface Preparation

Preferred use for the surface conditions below	Method
Removal of oils, greases, proteins (water-soluble, agents water-emulsive)	Steam jets with added wetting agents
Removal of old paint	Steam jets with added wetting agents and sand
Removal of old paint, heavy contamination on low-strength surface areas of the concrete, damage from the road salts	Sand-blasting, Water sand-blasting, High pressure water-jets
Removal of thicker old coatings from deeper areas with low surface strength, deep-reaching contamination	Flame-cleaning and mechanical cleaning
Removal of deep-reaching road salts and other contamination.	Grinding and mechanical cleaning

The optimum roughness of substrate (0.5-1.0 mm) can be achieved with sandblasting technique [SCD-1]. Protruding arises, remains from shuttering, dowels etc. must be removed. Visual check of the treated substrate surface for foreign matter and inclusions in the concrete is part of quality control.

3.2.2. Steel Surfaces

- Removal of grease, oil, rust and scale;
- Preparation - sandblasting;
- Watch for the formation of condensation (dew point);
- If not bonded immediately, the surface has to be protected with compatible corrosion inhibitor.

3.2.3. FRP Sheets

- Place FRP fabric on even surface (such as table).
- Check the material for possible damages, cracks etc.
- The CFRP strips have to be cut to proper length by metal saw or disk-cutter before application.
- Sika recommends to wipe clean with appropriate cleaner (e.g. acetone). This operation removes soiling as well as carbon dust. Cleaning should be continued until white cloth remains white.
- Dry CFRP laminate with a clean rag (Sika)

3.3. Mixing of Adhesives

- Consult technical data sheet for specific type of adhesive.
- Protect adhesive and components from direct sunlight.
- Maintain appropriate proportions of components. (parts by weight or parts by volume).
- Prepare only that quantity which can be used within its pot life.
- Premix each component in the original containers.
- Consult technical data sheet which component should be added to which. Generally hardening component is added to resin component.
- Mix the adhesive until uniform in consistency (uniform color). Use electric hand mixer for about 3 minutes or mix manually with the trowel or spatula. Mix with low speed so that as little air as possible is entrained (max. 500 rpm).
- Take care to scrape the sides of the pail during mixing.
- The pot life of the adhesive begins when the resin and hardener are mixed.
- To obtain longer workability and pot life the following suggestions can be followed:
  - Pot life is longer at lower temperatures. If necessary the adhesive components can be chilled to room temperature before mixing.
  - Adhesive prepared in smaller quantities has usually longer pot life.

3.4. Application to Structure

- If the system being used requires the primer (Tonen, Mitsubishi), apply according to commercial specifications.
- Apply the mixed adhesive onto the concrete with trowel, float or spatula to form a layer of required thickness.
- The adhesive has to be applied with great care to the concrete surface to ensure that all the voids are filled and no cavities are left.
- Apply the mixed adhesive onto the CFRP laminate to form a layer of required thickness.
- Within the open time of the adhesive, depending on the temperature, place the FRP onto the concrete surface.
- Press the laminate into the adhesive using the hard rubber roller until the adhesive is forced out on the sheet sides.
- Remove excess adhesive.
- Leave the applied layer undisturbed for at least 24 hours.
- Repeat the application process for desired number of layers. Maintain proper width of glue line.
- Apply coating for protective or aesthetic finish (optional).
- During the curing time of the adhesive heavy traffic loads should be regulated in order to avoid a possible diminution of the bond strength of the adhesive to the concrete surface.

3.5. Additional Limitations and Requirements

- Consider that FRP materials are easily damaged in transport or in the field by cutting, bending , and trampling.
- The minimum age of concrete must fulfill the requirements of a particular system. In most cases, it should be 21-28 days unless special curing and drying conditions are provided.
- Do not thin the adhesive unless specified. Solvents will prevent proper curing.
- CFRP material is a vapor barrier after cure.
- Minimum substrate and ambient temperature depends on type of adhesive used. It has to be checked with data sheet.
- In extreme solar radiation, most cold-setting epoxy adhesives experience a reduction of shear modulus and shear strength.

3.6. Safety Precautions and First Aid

Safety Precautions - Basic Measures.

- Consults the material safety data sheet for the adhesive before application.
- Epoxy resins can cause skin sensitization or irritation (dermatosis) after prolonged or repeated contact. They can also be eye irritant or even cause burns.

- High concentration of vapor may cause respiratory irritation. Therefore adequate ventilation is necessary. Overexposure may affect liver, kidney, and/or central nervous system effects.
- Use of standard precautions such as safety goggles and chemical resistant gloves is recommended. Cover hands with barrier cream before starting work.
- In any case avoid contact and limit exposure to a necessary minimum.
- In case of skin contact wash immediately and thoroughly with soap and water. Contact physician if symptoms persist.
- In case of eye contact wash immediately with plenty of water for at least 15 minutes. Immediately contact physician.
- In case of respiratory problems, remove person to fresh air. Contact physician if symptoms persist..
- In case of respiratory overexposure (excess of PELs) use the appropriate, properly fitted NIOSH/MSHA approved respirator.
- If sanded , possible exposure to crystalline silica (sand) dust may cause delayed lung injury and is listed as a suspect carcinogen by NTP and IARC. Use of appropriate dust protection is recommended.

<u>Safety Precautions - Additional Measures.</u>

- In case of spills or leaks, wear suitable protective equipment, contain spill, collect with absorbent material and transfer to suitable container. Ensure proper ventilation of the area.
- Uncured material can be removed with approved solvent.
- Cured material can only be removed mechanically.
- Dispose of in accordance with current, applicable local, state and federal regulations.
- Unused adhesives should not be discharged into drains, waterways or ground.
- Keep materials out of reach of children.

3.7. Commercial Systems - Comparison of Application Procedures

Table 3.2 provides the comparison of application procedures for the three main strengthening systems evaluated here.

Table 3.2 Comparison of Commercial Application Procedures

Procedure	Sika *SikaDur*	Tonen Forca Tow Sheet	Mitsubishi *Replark*
FRP Cleaning	Yes (required)	Yes (required)	Yes (required)
Concrete Water Jet	Yes (allowed)	Yes (experiment)	Yes (experiment)
Concrete Sandblasting	Yes (required)	Yes (experiment)	Yes (experiment)
Concrete Grinding	Optional	Yes (required)	Yes (required)
Priming	No	Yes	Yes
Putty (Filler) application	Yes SikaDur 41	Yes	Optional
Undercoating - 1st Resin Coating for 1st and further plies	Yes	Yes	Yes
Protective overcoating - 2nd Resin Coating	No	Yes	Yes
Finishing and Painting	Optional	Optional	Yes

3.7.1. Conditions of Service

Considerations of safety may be determined for the conditions of service of the strengthened structure:

- For whatever reason a laminate fails, it is necessary to ensure that the consequential damage can be assessed and no safety risks are incurred. According to U. Meier a post-strengthened structure should continue to have a total safety of 1.2 after laminate failure [EMPA-03].
- Recording of the condition of the structure requiring strengthening is important. Dimensions, quality of the existing construction, materials and ambient conditions must be established.
- The fact that CFRP laminates have no plastic deformation reserve, must be allowed for in the calculation of the load bearing capacity with an appropriate reduction factor less than that for steel members. The engineer can maximize the ultimate load of the full system by appropriate selection of the strengthening level in the critical sections.
- Bending failure in a conventional reinforcement concrete structure is generally preceded by serious deformation and wide cracks, but advance warning of imminent laminate failure in a post-strengthened structure has to be obtained in some other way.

CHAPTER 4
REVIEW OF RESEARCH AND FIELD APPLICATIONS

4.1. Introduction

Different types of unidirectional and bi-directional FRP sheets made of Carbon (CFRP), Glass (GFRP) or Aramid (ARFP) fibers are used both in research and applications for strengthening and repair of existing structures. They are used as a substitute for steel plates. FRP sheets possess a number of features that make them suitable for such applications:

- good mechanical properties;
- low volume to weight ratio;
- immunity to corrosion;
- no need for the formation of joints (practically unlimited delivery length of FRP).

The research projects carried out at different organizations were aimed at various aspects of strengthening concrete structures:

- bonding of fiber-reinforced plastic composite plates or fabrics to reinforced concrete and prestressed concrete beams to improve flexural stiffness and strength;
- wrapping of concrete beams with composite fabric/epoxy jackets to provide additional shear strength;
- wrapping of concrete columns with fiberglass/epoxy jackets to provide the additional flexural stiffness and strength, in particular for seismic retrofit;
- confinement of concrete columns to increase column ductility as well as load capacity.

Typical applications of FRP for strengthening RC and PC beams for flexure and shear are given in Figure 4.1.

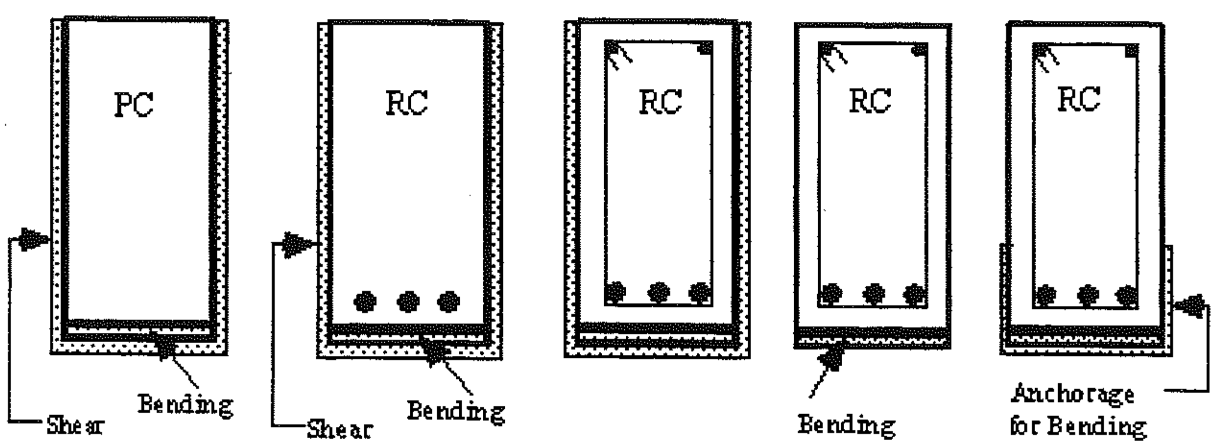

Figure 4.1 Typical Applications of Glued-on FRP Composites for Shear

PC = Prestress Concrete, RC = Reinforced Concrete

A review of research issues on the use of FRPs can be found in references: [SOA-1], [CA-1], [XX-7], [XX-12]. This report is devoted to the first and second aspects - strengthening of the beams in flexure. A summary of the reviewed research on strengthening of beams (structures) in flexure is presented in Table 4.1 for quick reference. The literature about FRP addresses the following topics:

- Structural behavior (Flexural behavior; Shear, Fatigue, Bond);
- Durability (freeze-thaw, chemical aggression, low temperature, UV light, fire resistance);
- Field applications and demonstrations.

The following sections give summary of literature for each of the topics with division into categories. Field demonstrations are in most cases performed on real structures and therefore no failure information is provided. The results are limited to visual and electronic monitoring of the structures.

4.2. Structural Behavior

This section reviews structural features and significant issues of this new construction technology. It covers different strengthening systems (CFRP, GFRP AFRP, steel plates) and their comparison.

The structural behavior of laminate bonded beams depend on many variables. Successful implementation of the technology is strictly dependent on maintaining the highest possible standards and compliance with the requirements of a particular strengthening system. The following list of factors to consider, as taken from [X-12], is not exhaustive, but indicates the complexity of the issue and the scope of the literature.

- Geometry of the beam;
- Mechanical properties of the concrete of the beam (compressive strength, modulus of elasticity);
- Geometrical properties of the FRP laminate: (thickness, area);
- Mechanical properties of the FRP laminate (tensile strength, modulus of elasticity);
- Type of composite (Carbon - CFRP, Glass - GFRP, Aramid - AFRP);
- Type of primer and adhesive;
- Surface preparation procedures;
- Composite application procedures;
- Loading (static, dynamic);
- Beam reinforcement (under-reinforced, over-reinforced, prestressed);
- Beam state of cracking;
- Exposition to environmental actions: (corrosion, weather, freeze/thaw, sun light, aging);
- Bond of the interface between the FRP plate and concrete.

Table 4.1 Summary of FRP Literature by Organization or Research Team

Institution	Research Team	Type & Grade of FRP (other reinf. systems), Adhesives	Laboratory Tests	Field Tests, Demonstrations, Applications	Reference Code	References
Sika		CarboDur			SCD	SCD
Tonen Corporation, H.S. Kliger & Associates, Structural Preservation Systems	Akira Kobayashi , Ohori N., Hiroyuki Kuroda, Hiroyuki Yoshizawa , Howard S. Kliger, Jay Thomas,	CFRP - Forca Tow Sheet	Bonding strength tests, Concrete beams, 4-point bending, Flexural capacity, Fatigue of slabs	RC-slab in highway bridges (FTS-1-3).	FTS	FTS-1-6
Mitsubishi , Craig Ballinger & Associates	Moriyasu Nakamura Hiromichi Sakai Kensuke Yagi Tsunego Tanaka Craig Ballinger T. Maeda T. Hoshijima	Replark, High Modulus CF - E65, Inter Modulus CF - E24	Concrete beams, 4-point bending, Flexural capacity		MRK	MRK-1-3
Ohio DOT, Wright Patterson Air Force Base	Steven E.Morton John Mistretta Karen R. Schaefer	AS4C/1919 Hercules Inc.	Tensile test of fiber material, Concrete & steel adhesive bond test, 4-point bending of Concrete & steel beams, Freeze thaw tests, Fatigue tests	Butler County Ohio - RC bridge	OH	OH-1-2
Ohio State University, Wright Patterson Air Force Base	Muszynski L.C., Sierakowski R.L.	AS4C/1919 Hercules Inc.	Concrete beams, Fatigue tests, Three-point bending		OH	OH-3
Deleware DOT, University of Deleware	Michael Chajes William W. Finch Ted F. Januszka Dennis R. Mertz Chao Hu Cory A.Farschman Theodore A.Thomson	Carbon Fiber, E-Glass Fiber, Aramid Fiber (Kevlar), CFRP - Tonen, SikaDur 32	Concrete beams, 4-point bending, Flexural capacity, Freeze/thaw tests, Dry/wet tests, Fatigue tests	Foulk Road Bridge #26, Wilmington, Delaware.	DE	DE-1-3

Institution	Research Team	Type & Grade of FRP (other reinf. systems), Adhesives	Laboratory Tests	Field Tests, Demonstrations, Applications	Reference Code	References
Swiss Federal Laboratories for Materials and Testing, EMPA	Urs Meier Gregor Schwegler Martin Deuring	GFRP plates CFRP plates T300 T700 fibers	Concrete beams Bending, Flexural capacity Freeze-thaw tests Fatigue tests Durability (Fire tests)	Kattenbush bridge Ibach bridge, Switzerland	EMPA	EMPA-1-12
University of Arizona	Hamid Saadatmanesh Mohhamad R Ehsani	GFRP plates CFRP plates	Prestressing of plates before testing 4-point bending Analytical models Testing of epoxies		AZ	AZ-1-11
RUTGERS The State Univ. of New Jersey	P.Balaguru Stephen Kurtz Jon Rudolph	Carbon Fiber T300, Geopolymer	4-point bending, Concrete beams (4 pc), Flexural strength		RNJ	RNJ-1-2
West Virginia University, Constructed Facilities Center	Hota V.S. GangaRao, Salem S. Faza, Ever J. Barbero	CFRP - Tonen FTS-C1-20, Steel plates	4-point bending Concrete beams (21pcs), Flexural strength		WVU	WVU-1-3
Universite de Sherbrooke (Canada)	M'Bazaa I., Missihoun M., Labossiere P.,	CFRP - Tonen FTS	4-point bending Concrete beams (8 pcs), Flexural strength		XX	XX-1
Naval Facilities Engineering Service Center, University of California	Malvar L.J., Warren G.E., Inaba C.	CFRP - Tonen FTS-C1-20	3-point bending, Concrete beams (6 pcs), Flexural strength, Shear strength. Square slabs (6pcs), Punching shear		XX	XX-2-3
King Fahd University of Petroleum and Minerals	Sharif A., Al-Sulaimani G.J., Basunbul I.A., Baluch M.H., Ghaleb B.N.	CFRP plates	Concrete beams (10 pcs), 4-point bending		XX	XX-5-6
University of Wyoming	Prof. Charles Dolan				UWY	
University of California	Prof. Frieder Seible				CA	
New Jersey Institute of Technology	dr William Spillers					
Pennsylvania State University	Antonio Nanni				PSU	
Lawrence Technological Institute	Prof.Nabil F.Grace					
Georgia Institute of Technology	Abdul-Hamid Zureick				GAT	
FDOT Florida DOT	Mohsen Shahawy				FL	FL-1

Institution	Research Team	Type & Grade of FRP (other reinf. systems), Adhesives	Laboratory Tests	Field Tests, Demonstrations, Applications	Reference Code	References
CDOT California DOT	Mosen Soltan				CA	CA-1-2
US Corps of Engineers	Edward O'Neil				USCE	
US Corps of Engineers	Jonathon Trovillion					
FHWA Federal Highway Administration	Eric Munley				FHWA	FHWA
Catholic University	Lawrence Bank					

The bond between the composite and concrete substrate has a significant role in structural characteristics of the reinforced members. The influences of bond can be classified as follows [CA-1]:

Internal influences:	Chemical Activity Electrochemical activity Alkali content and pH level Stress Moisture infiltration Transport of solutions, salts
Interfacial influences:	Moisture entrapment Moisture diffusion Selective transport of chemicals Thermal and elastic mismatch
External influences:	Humidity Moisture Temperature Temperature Cycling (daily, seasonal, annual) Aggressive natural and manmade agents UV light Oxygen (related to steel)

The dominant mechanism of bond in FRP reinforced structures is by mechanical interlock. The rough and deep surface topography created through surface abrasion creates a structural morphology that allows the resin to penetrate into irregularities forming a strong interfacial layer [CA-1].

4.2.1. Modes of Failure of Post-strengthened Beams

There are different modes of failure of beams post-strengthened with FRP. Depending on the particular design of the beam the failure of the FRP system can cause immediate failure of the member or influence its reliability. The modes of failure depend on the factors listed in the previous section. Some modes are more probable than other. The possible failure modes are:

a. Failures of concrete
   - (CC) - Failure of concrete compressive zone when the maximum concrete compressive stress is reached.
   - (CS) - Vertical shear failure of concrete.
   - (CP) - Punching shear.

- (CD) - Failure of concrete cover (delamination in concrete)

b. Failures of Reinforcing Steel
   - (RT) - Failure of reinforcing bars - yield strain exceeded.
   - (RF) - Fatigue failure of reinforcing bars - fatigue strength exceeded.

c. Failures of FRP laminates
   - (FT) - Failure of FRP laminate when tensile strength is reached.
   - (FD) - Inter-laminar failure of the laminate (delamination in FRP).
   - (FC) - Failure of FRP material caused by transverse reciprocal displacement of crack edges at the bottom of the beam.

d. Failure in adhesive surface
   - (BF) - Adhesion failure (FRP-concrete surface).

The above codes for failure modes are utilized throughout the report. The examples of the most typical failure modes are illustrated in Figure 4.2.

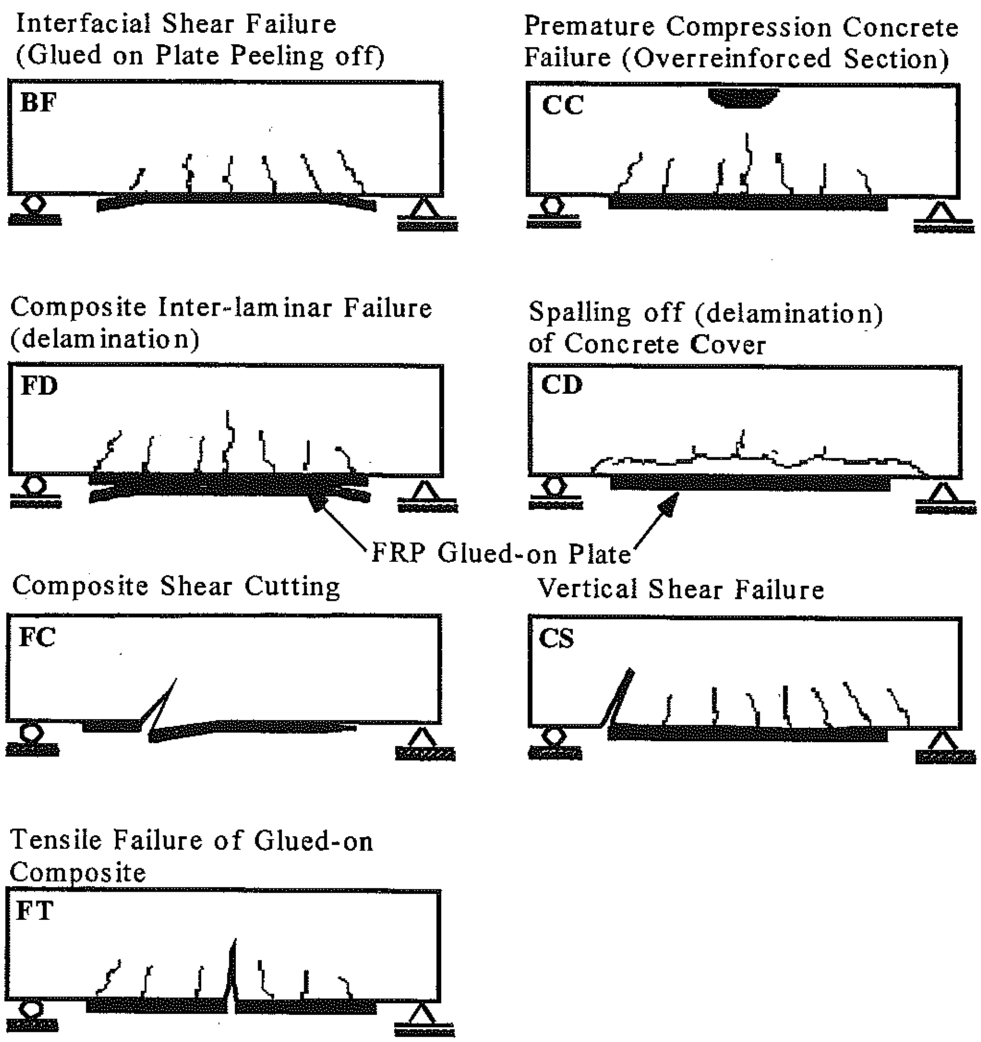

Figure 4.2 Typical Failure Modes of RC and PC Beams Strengthened with FRP

4.3. Flexural Behavior

FRP sheets used for strengthening the flexural capacity of concrete elements reveal their great potential thanks to their strength and stiffness. This results in a relatively low cost of repair.

In the majority of flexural applications, unidirectional sheets prove to be good performers in strengthening the mid span section of reinforced and prestressed concrete beams. In cases of complex states of stress such as bending combined with shear, two or multidirectional composite fiber laminates have to be used

The following sections summarize the experimental works focused on the flexural behavior of FRP strengthened concrete elements The information covered is classified according to its source.

4.3.1. FTS - Tonen Corp., H.S. Kliger & Associates, Structural Preservation Systems

References: [FTS-1][FTS-2]

The experimental program was carried out to investigate the flexural strength of reinforced concrete beams post-strengthened with CFRP. The unidirectional fabrics were glued to the tension face of the beams. Total of ten beams were tested. Forca Tow Sheet FTS-C1-30 & FTS-C5-30 was used as the bottom strengthening. One of the beams was strengthened with steel plate and one control beam had no external strengthening. The wrapping with FTS at 90 deg angle was done in some beams to investigate the influence of anchoring on failure mode. It was observed that the Tow Sheet increased the yield load in the beam in comparison to control case, regardless of the sheet angle and/or number of plies applied. The increase of yield load varied from 91% to 158%, for one and three plies of fabric respectively. The mode of failure depended on the particular beam. In the beams having the anchoring by 90 deg wrapping of longitudinal fabrics no peel-off effect was observed. Due to the use of this type of anchoring the ultimate strength of beams was maximized and increased by 207% over the bare beam. Other samples exhibited the peel-off in cohesive layer of concrete. The comparison of the experimental results with theoretical predictions proved good agreement.

Reference: [FTS-5]

The experimental program was carried out to investigate the flexural strength of reinforced concrete beams post-strengthened with CFRP. The main purpose was to investigate the influence of surface preparation on the effect of strengthening. The unidirectional fabrics were glued to the tension face of the beams. Total of eight beams were tested. In general CFRP increased yield load of beams in comparison to unreinforced (control) beam. The maximum load is greatly influenced by concrete surface treatment. The beams treated with water jet performed significantly better than the beams treated with sander. The increase of maximum load due to CFRP equaled accordingly 70-80% for the beams treated with water jet and 30-40% for those treated with sander. It was concluded that water jet treatment is a very effective way of surface preparation.

4.3.2. MRK - Mitsubishi (Replark), Craig Ballinger & Associates

Reference: [MRK-1]

A series of bending tests was carried out on rectangular reinforced concrete beams in four point setup. Two different types of CFRP sheets were used: High-Modulus E65 and Inter-Modulus E24 (general-purpose). The unidirectional fabrics were glued to the tension face of the beams. One and two-layer strengthening was selected. Strain gauges monitored the deformation of specimens. The tests revealed that in comparison to non-reinforced beam, the crack occurrence load was improved by 32% with one ply and 72% with two plies for the Inter-Modulus CF, and by 88% with one ply and 120% with two plies of High-Modulus CF. Comparing to non-reinforced beam, the maximum load was improved by 40% with one ply and 71% with two plies for the Inter-Modulus CF, and by 42% with one ply and 82% with two plies of High-Modulus CF. Flexural rigidity increased by 24% with one ply and 31% with two plies for the Inter-Modulus CF, and by 50% with one ply and 91% with two plies of High-Modulus CF. The sheet with the higher Young's modulus has higher reinforcing effect. The comparison of calculation methods and results of computations proved good agreement. It was concluded that the procedure for design of RC-beams with CFRP reinforcement should assume the following order of failure:

- Yield of tension reinforcement,
- CFRP sheet breakage,
- Crushing failure of concrete.

4.3.3. AZ -University of Arizona

References: [AZ-01][AZ-03]

Saadatmanesh, H and Ehsani, M.R. presented the experimental study of 5 rectangular beams and one T-beam strengthened by gluing GFRP plates to their tension flanges and tested under 4 point bending. Based on the results of an earlier study, a rubber-toughened epoxy with a consistency similar to that of cement paste was used for strengthening of the beams. As a result of this study, it was concluded that a significant increase in flexural strength can be achieved by bonding GFRP plates to the tension face of reinforced concrete beams. The gain in ultimate flexural strength was more significant with lower steel reinforcement ratios. In addition, plating reduced crack size in the beams at all load levels but also reduce the ductility of the beams. The successful application of this technique requires careful preparation of the concrete surface and selection of a tough epoxy. Finally, calculated load-deflection curves were obtained using the strain compatibility method. Compared with the measured load-deflection curves, they correlated well.

Reference: [AZ-2]

Char M.S., Saadatmanesh, H and Ehsani, M.R conducted a parametric study of the flexural strength of concrete girders externally prestressed with epoxy bonded fiber reinforced plastic (FRP).Variables considered are type of FRP plate, area of plate, and concrete compressive strength. The girders were externally prestressed with FRP plates epoxy bonded to the tension face of the girders, while the girders were held cambered by means of upward jacking forces. As a final step, a design example is presented showing how this technique can upgrade the load carrying capacity of a typical concrete bridge originally designed for H15 loading to that of HS20 loading.

For this study, analytical models are presented to calculate moment and curvature for concrete girders externally prestressed with GFRP plates throughout the entire range of loading up to failure. Conclusions of this study are :
- Bonding a FRP plate to the tension face results in a significant increase in the moment carrying capacity of the girder.
- Increasing the area of the plate from 4 to 8 in further increased the moment carrying capacity of the section.
- The behavior of the beam with camber is similar of the one with no camber. The moment capacity of the beam remains the same.
- The higher tensile strength of the CFRP plate resulted in a significantly greater increase in the moment capacity compared to that for the GFRP plate. However, the ductility of the beam is reduced due to the greater stiffness of the CFRP plate.
- On the design example, stresses in beams strengthened with GFRP and CFRP plates without camber are slightly higher than the allowable stresses as provided by AASHTO specifications. The externally prestressed beams satisfy the AASHTO allowable stress requirements.
- Beams that have been strengthened for flexure may fail in shear. Flexural strengthening may require that the shear strength of the beam be increased as well.

4.3.4. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-1]

At the University of Delaware the tests were performed on 14 rectangular under-reinforced concrete beams. The experiments were aimed at evaluation of the effect of externally bonded, composite FRP sheets on the beams flexural behavior. Different types of FRP were used (CFRP, GFRP, AFRP). The four-point bending setup was used with load increased monotonically to failure. The general flexural behavior of the composite-fabric-reinforced beams was similar, although the flexural stiffness and final mode of failure varied depending on the used type of fabric. Beams reinforced with fabrics displayed less ductility compared to the beams reinforced with steel alone, however they exhibit some measure of ductility prior to failure. All the externally reinforced beams showed significant increases in ultimate flexural capacity, as compared with control beams, within the range 33.6 to 57%. Also the average stiffness of the specimens increased within the range 45 to 53.2%. Parallel to experiments the analytical model for calculation of beam capacity was developed giving the results close to test values. The results of the study indicate that the externally applied reinforcement can be effectively used to rehabilitate or strengthen the concrete beams. The analytical methods to describe their behavior are available. The bond between the fabric and the concrete, combined with the additional anchorage ensured the failure of either the concrete in compression or the fabric in tension.

4.3.5. RNJ - RUTGERS The State Univ. of New Jersey, GEOPOLYMERE

Reference: [RNJ-1]

The tests of flexural strength of beams carried at the RUTGERS University of New Jersey were aimed and checking the application of carbon fiber and geoplymer adhesive. The unidirectional fabrics were glued to the tension face of the beams. Four specimens were tested having different thickness of the CFRP layer. The design of the beams was such as to compare the results with the tests carried out by M'Bazaa I., Missihoun M., Labossiere P. (ref. XX-1). The results indicate that geopolymer provides excellent adhesion both to concrete surface and interlaminar planes of fabrics. All the specimens failed by tearing of fabrics. Geopolymer adhesive is fire-resistant (up to 1000°C), does not degrade under UV light and is chemically compatible with concrete. This inorganic polymer is a water-based alumino-silicate. Its performance is better than organic polymers. It does not require any special protective equipment other than gloves. The stiffness of the beams increased with the number of CFRP layers as expected. The beams strengthened with CFRP exhibited more dense crack pattern with smaller widths as compared with control beam.

4.3.6. OH - Ohio Department of Transportation, Wright Patterson Air Force Base

Reference: [OH-2]

A series of bending test was carried at Wright-Patterson AFB to study carbon fiber plates glued to bottom of beams as reinforcement. A commercial graphite/epoxy system (AS4C/1919 from Hercules Inc.) was used. Plates having both unidirectional and two directional lay-ups of the fibres were applied to beams. Forty specimens were tested. Four configurations (types) of composite plates were chosen:

A - single plate w/o lap joints,
B - single lap joint in maximum moment region,
C - two single lap joints outside of maximum moment region,
D - two coupler in locations as in conf.C.

The adhesive surfaces were prepared prior to application of reinforcement. Vacuum Bag was used to provide good interfacial bond of the plates to beam surface. The failure of lap joints in configuration B proved its inadequacy. Configuration D introduces smaller stress concentrations in comparison to type C lap joint. No failures were noted in type D joint and it was selected for use in planned field demonstration. The results of the tests indicated the substantial increase of beam stiffness and strength. Good agreement was achieved in comparing the test results with the theoretical predictions. The results of the experiment were aimed at gaining experience before the field demonstration on a bridge in Butler County in Ohio.

4.3.7. WVU - West Virginia University, Constructed Facilities Center

Reference: [WVU-1]

Twenty one reinforced concrete beams were tested using CFRP and steel plate strengthening systems. The beams were preloaded up to cracking of concrete and close to yielding of steel Reinforcement prior to application of strengthening system. The four-point bending setup was selected with 9-foot span and 3x3-foot force distance. The test setup was instrumented to measure the strains and deflections. The load was applied monotonically at constant rate. The use of CFRP increased the load carrying capacity of the beams. The average strength increase equals 57%. Steel plate strengthening has not resulted in any significant increase (6%). The effect of wrapping configuration is not evident between different groups of wrapped beams. The beams strengthened with CFRP failed by crushing of concrete between the points of load application, without any delamination of carbon-fiber wrap. High strength to weight ratio of CFRP resulted in a better physical integration and performance.

4.3.8. EMPA - Switzerland: Swiss Federal Laboratories for Materials Testing and Research

Reference: [SOA-1]

Meier (1987) reported the use of thin CFRP sheets as flexural strengthening reinforcement of concrete beams. He showed that CFRP can replace steel with overall cost savings in the order of 25 %. Kaiser (1989) load tested CFRP composites on full scale reinforced concrete beams and showed the validity of the strain compatibility method in the analysis of cross sections. It was suggested that inclined cracking may lead to premature failure by peeling-off of the strengthening sheet (BF type of failure). The study included the development of an analytical model for composite plate anchoring, which was shown to be in agreement with tests results.

Reference: [EMPA-1]

Table 2.4 from 2.1.4, [EMPA-1], list criteria which specifically relate to the use of composite material as a post-strengthening material for structures and applies particularly to prestressed laminates. From this table, it is clear that Carbon Fiber Reinforced polymer Composites most closely fulfill requirements for the post strengthening of structures.

Considering the case when the plate is not prestressed before bonded, the modulus of elasticity of the plate material is of great significance, because only stiff plates are able to relieve the stresses in the existing internal steel reinforcement. Therefore, CFRP has a noticeable advantage over GFRP.

The experimental work showed that the possible occurrence of shear cracks may lead to peeling off of the strengthening composite. Thus, shear crack development represents a design criterion. The following failure modes were observed in the flexural loading tests:

Failure of concrete compressive zone when the maximum concrete compressive stress is reached.(Failure type CC).

The CFRP composite failed suddenly with a sharp explosive snap; the impeding failure was preceded well in advance of the failure by cracking sounds.

Vertical shear failure of concrete (Shearing of the concrete in the tensile zone, failure type CS)

Fatigue failure of reinforcing bars (Failure type RF).

Inter-laminar failure of the laminate.

Interlaminar shear within the CFRP laminate (Failure type FD)

Adhesion failure FRP-concrete surface (Failure type BF)

4.3.9. XX - Other Research Institutions

Reference: [XX-1]

At the Universite de Sherbrooke (Canada) the experimental program was carried out to investigate the flexural strength of reinforced concrete beams post-strengthened with CFRP. The unidirectional fabrics were glued to the tension face of the beams. Total of eight beams were tested. One of the test beams (PC) was not strengthened with CFRP and it served as the control specimen. The Tonen Forca Tow Sheet composite was applied to bottom of the beams. The deflections of the beams were monitored using LVDTs in mid-span. The load was applied monotonically up to failure. The crack pattern in the beams post-strengthened with CFRP was more dense and uniform with smaller widths in comparison to control beam. Final load occurred under the load at least 56.4 % larger than the control beam. In all cases the beams failed due to sudden delamination of composite (peeling-off failure). Also the stiffness of the CFRP beams increased substantially what resulted in the lower deflections at failure. The placement of CFRP at different small angles had no particular influence on strength and stiffness. The analytical models used in the work provided the reasonable approximation of the flexural behavior.

References: [XX-2] [XX-4]

At the US Navy Facilities Engineering Service Center the experimental program was carried out to investigate the flexural strength of under-reinforced concrete beams strengthened with CFRP. The unidirectional Tonen FTS-C1-20 fabrics were glued to the tension face and the sides of the beams in different way. Total of six simply supported beams were tested. The load was monotonically increased up to failure. The tests were instrumented with strain gages and LVDTs. The experimental results were compared with the anaytical computations. The results of the study indicate that the flexural strength can be significantly increased by bottom CFRP but the beams will fail in shear. Additional vertical tows at beams sides can provide sufficient shear strength and revert the mode of failure to bending. Up to 90% increase of bending strength was achieved.

References: [XX-5] [XX-6]

At King Fahd University of Petroleum and Minerals the experimental program was carried out to investigate the flexural strength of reinforced concrete beams strengthened with CFRP. Total of ten simply supported beams were tested. Prior to application of FRP the beams were preloaded up to approx. 85% of their ultimate capacity to introduce initial damage. The fiber glass plastic FRP with three layers was glued to the tension face of the beams. Four different configuration of strengthening were chosen:

A. Plates bonded to beam soffit;
B. Conf. A + anchorage with steel bolts;
C. Conf. B + side FRP plates;

D. Jacket plates bonded to soffit and sides of beams.

The rehabilitated beams were tested up to failure. The conclusion from the tests were as follows:

1. The shear and normal stresses at the plate curtailment increase with increasing FRP plate thickness, leading to premature failure by plate separation and concrete rip-off.
2. Steel anchored bolts eliminated plate separation and the curtailment zone for large-thickness plates. The repaired beams failed due to diagonal tension cracks.
3. Jacket FRP plates provided the best anchorage system to eliminate plate separation and diagonal tension failure, and develop the flexural strength of the repaired beams.
4. Repaired beams provided enough ductility despite the brittleness of FRP plates, indicating effectiveness of FRP for external strengthening.

Reference: [XX-37]

Experimental program was conducted at Leheigh University (Pennsylvania) in order to test the flexural behavior of concrete beams strengthened with different glued-on systems. A series of 16 under-reinforced beams was tested. Two beams without strengthening served as control samples. The following different plates were used for the remaining beams:

1. A molded fiberglass standard pultruded fiberglass sheet;
2. 0/90 deg molded fiber reinforced plastic;
3. A molded fiberglass standard pultruded fiberglass channel;
4. 0/90 deg 65% GFRP / 35% CFRP;
5. A spring-orientation Glass FRP;
6. 0/60 deg Carbon FRP;
7. 0/90 deg Carbon FRP;
8. Unidirectional Aramid FRP;
9. Mild steel plate.

A standard ready-mix concrete was used. Before the application of FRP the adhesive surfaces were prepared by sandblasting and high pressure water washing.
Different anchorage systems were used:

- FRP extended up to support;
- Partial height bonded angles;
- Full height bonded plates connected by bonded angles.

The experiment proved the increase of strength and stiffens of beams with FRP from 19 to 99% over control samples. All FRP plates demonstrated brittle behavior. The role of anchorage type was confirmed by the test results.
A computer model was developed which allowed for satisfactorily accurate prediction of the beams flexural strength. To fully validate the model more tests are necessary.

4.4. Shear Behavior

Shear performance of the FRP strengthening systems is a major research concern due to the following factors:

- Multidimensional state of strength, requiring the use of multidirectional FRP composite laminates.
- Limited development lengths of the FRP laminate may require additional anchorage provisions (e.g. bolted connections).
- Limitations of access to the sides of the girders (e.g. box girders in a slab bridge)

The available information confirms that FRPs can be successfully used for increasing the shear capacity of concrete elements provided anchorage issues are properly resolved. The following sections summarize the experimental investigation dealing with the shear behavior of FRP strengthened concrete elements. The information covered is classified according to its source.

4.4.1. SOA - State of the Art Reports & Review Documents

Reference: [SOA-1]

External shear reinforcement in the form of bonded FRP overwrap has been applied to beams with insufficient shear strength. Tests conducted by Rider, 1993, have indicated that this procedure provided sufficient shear resistance to allow full development of the flexural capacity of the beam.

4.4.2. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-5]

The paper presents the comparison of effectiveness of a variety of woven composite fabrics with different strengths and stiffness (Aramid, Glass and Graphite FRP) in application to shear strengthening. Also included is the study of the orientation of bonded fabric on its shear performance. The tests were carried out on a total of twelve T-beams designed according to ACI specifications. The beams were underdesigned in shear compared to their flexural capacity. Woven composite fabrics with fibers oriented at both 0 and 90 deg, with equal distribution in each direction were employed. The following fabrics were used:

- Plain-weave Aramid fabric;
- Crowfoot satin weave E-glass fabric;
- Plain-weave Graphite fabric.

The beam surface was prepared prior to bonding FRP. It was mechanically abraded until the layer of laitance was removed, followed by air-blasting to remove any loose particles. In application of FRP the vacuum bag was used in order to remove the excess of adhesive and obtain best possible bond. Four-point bending setup was used. Three control beams had no FRP reinforcement for comparison. All tested beams failed in shear, with strengthened beams having significantly higher ultimate load. All beam experienced brittle failure mode with significant tension cracking in constant shear span. Wrapping with FRP increased the carrying capacity by 60-150% over control beams. The order of beam capacity was as follows: Control, Aramid (82.7% increase), E-glass (87.9% increase), Graphite (91.1% increase). The beams wrapped with Graphite with 45/135-deg orientation displayed a 125.3% increase in strength. The theoretically computed shear strengths for control beams were, on average, 28% lower than the measured values. An analytical procedure was developed to take into account the share of FRP. The shear capacity could be estimated with 9.1-13.2% accuracy. The predictions for beams with FRP proved more accurate than for the control beams. All predicted nominal shear capacities are conservative.

4.4.3. XX - Other Research Institutions

Reference: [XX-32]

Tests were carried out on three reinforced concrete beams to investigate the effect of FRP wrapping of the RC beams having the floor slab. Two of the beams had rectangular cross-section, one of them strengthened with FRP, and served as control beams. Third beam with T-shaped cross section was strengthened with FRP by anchoring the sheet at its ends into the underside of the flanges. Tonen Forca Tow sheet was used having the orientation perpendicular to beam axis. The Anti-symmetric Moment method was selected as the test setup. Comparable force-deflection diagrams and ultimate forces for both FRP-strengthened beams proved, that such methods of strengthening can be effectively used for beams with floor slabs. A method of analytical prediction of shear capacity using the so called "converted shear reinforcing bar ratio" was discussed.

Reference: [XX-33]

At University of Manitoba tests were carried out on four I-shaped beams of AASHTO design to investigate the shear characteristics of FRP reinforcements. The purpose of the tests was to collect the data necessary to strengthen the twin five-span continuous precast pretensioned concrete bridge. In the experiment, the model scale of 1:3.5 was assumed in regards to all applicable dimensions. Three types of CFRP were used in six different configurations. First beam served as control specimen. The beams were designed to carry the same shear stress as the ones in the structure. The beams were tested at each end in order to determine the most effective strengthening scheme. The following FRP strengthening systems were chosen:

- Beam No.2 - Forca Tow Sheet - Tonen Corporation,
- Beam No.3 - Replark - Mitsubishi,
- Beam No.4 - Tyfo™ S Fibrwrap™ - Hexcel Fyfe Company.

The following surface preparation techniques were used:

- Beam No.2 - Grinder, wire brushing, high pressure air;
- Beam No.3 - High pressure water blasting;
- Beam No.4 - High pressure water blasting;

The configuration of FRP wrapping was as follows:

- Beam No.2 - 1st end - vertical wrapping, 2nd end - vertical + horizontal wrapping;
- Beam No.3 - 1st end - 45 deg wrapping, 2nd end - 45 deg + vertical wrapping;
- Beam No.4 - 1st end - 45 deg wrapping, 2nd end - 45 deg + horizontal wrapping;

The non-symmetric test setup was used which allowed to test the ends of each beam in independent run. The strains were monitored using data acquisition system.

The results of the tests proved that the presence of FRP reinforcement improved the ultimate shear capacity of the beam in the following amount as compared to control beam:

- vertical wrap - 10% widely spaced sheets, 17% closely spaced sheets;
- horizontal + vertical wrap - 34%
- diagonal 45 deg wrap - 26%
- diagonal 45 deg + vertical wrap - data N/A
- diagonal 45 deg + vertical wrap - data N/A

It was concluded that the effectiveness of FRP overall effectiveness of diagonal wrapping for shear is higher than the horizontal/vertical combination.

Reference: [XX-31]

The investigation was done in order to evaluate the effect of shear strengthening of precast reinforced concrete bridge girders using CFRP sheets. Three hat-shaped beams salvaged from a demolished bridge were used. Uni-directional CFRP sheets were bonded to vertical faces in various arrangements. The sheets were applied continuously throughout the shear span. Either one or two layers of CFRP were bonded to the webs. One layer was oriented vertically. The second, if used was oriented horizontally. The members were tested up to failure. Four-point bending setup was chosen. Both strains and deflections were monitored throughout the testing. All the beams were tested twice, with the ends which failed first repaired before second run. Shear failure was governed by the strength of concrete rather than the CFRP material. The experiments indicated that anchorage is a key consideration. All the failure modes were by diagonal tension cracking. The effect of the CFRP was noticed once the cracks began to open up, at which time the sheets began to influence the behavior of the member. In general bond between CFRP and concrete was generally excellent. Some debonding took place in areas where the concrete surface was uneven. The horizontal CFRP sheets did not have significant influence on shear capacity. The results showed the increase in shear capacity of between 21% and 55% in comparison to unreinforced control beam. CFRP provides the crack control when glued to the surface of concrete members. The most efficient is the orientation perpendicular to crack line. If cracks are pre-existing, CFRP sheets may easily be applied in the optimum orientation. One of the possible analytical method of taking the CFRP strengthening into account is by using the truss model analogy, where CFRP sheets are treated the same way as stirrups. The truss model analogy was successfully modified to predict the strengthening effect of CFRP sheets. The use of this analytical procedure needs further verification for other types of sections and strengthening layouts.

Reference: [XX-43]

The experimental program was carried out at King Fahd University of Petroleum and Minerals to investigate the shear capacity of concrete beams strengthened with Fiberglass Plate Bonding. The beams were designed to have the shear deficiency in shear capacity; thus shear failure was the dominant mode of failure. Sixteen RC beams were tested The beams were divided into four main groups according to the FGPB shear repair scheme:

- Group C - control beams having no shear repair;
- Group S - beams repaired by shear strips;
- Group W - beams repaired by shear wings;
- Group J - beams repaired by U-jackets.

Each of the groups was subdivided into two subgroups:

- Subgroup O - beams having no flexural repair;
- Subgroup P - beams repaired in flexure by bonding a 3-mm thick flexural plate to the soffit of the beam to upgrade its flexural strength.

Prior to repair the beams were damaged to a predetermined level defined subsequent to testing of the control beams, which were unrepaired. Four-point bending setup was used. The tests were monitored with strain gauges and LVDTs. The tests revealed that the shear capacity is almost identical for both strips (Group S) and wings (Group W). Shear repair by jacket (Group J) is better than by strips or wings, thanks to good anchorage at the bottom of the beam, which prevents premature peeling failure. Thanks to it the beams with jackets failed in flexure. beams from groups S and W failed in shear. All of the repair methods restored the stiffness of the beams degraded during the preloading stage.

4.4.4. EMPA - Swiss Federal Laboratories for Materials Testing and Research

Reference: [EMPA-1]

A new method, which allowed the effective strengthening of the areas where the shearing force is present, was developed and tried out in several tests. (see figure 5 EMPA-1) The inner stirrup reinforcement is supplemented by a stressed or limply applied external strengthening component manufactured from advanced composite materials; this can be braided or unidirectional in form. The prestressing material is wrapped around the cross-section on one side and is anchored in the compression zone on the opposite side.
The shearing effect is influenced by many parameters and it can therefore be predicted only approximately. A vertical offset on the concrete surface can lead to bending forces

in the laminate which can exceed the concrete strength limits. Local unevenness can result in the shearing effect even at small loads, particularly in the case of thin, initially unstressed composites. Therefore CFRP should have a minimum thickness equal to 1 mm.

When a crack opens a vertical offset can occur. The shearing effect of a CFRP composite due to a vertical offset is dependent on the following parameters: loading (moment, axial force and shear force), geometry (Concrete, steel reinforcement., CFRP composite), mechanical properties (concrete, CFRP composite, rebars), geometry of the crack (micro and macro roughness, crack width, offset in the vertical direction) and the maximum elongation of the composite due to external forces.

The loading capacity of systems strengthened with initially unstressed laminates may be reduced if shearing is responsible for the failure.

4.5. Fatigue Properties

The fatigue properties of the FRP strengthening system are dictated by the characteristics of the fibers, type of matrix as well as the adhesives. Experimental results [EMPA 1, EMPA 11] showed that Carbon Fiber present an outstanding fatigue performance compared with Glass and Aramid. Therefore, CFRP sheets are particularly suitable for fatigue load conditions.

The following passages summarize the experimental works focused on the fatigue behavior of FRP strengthened concrete elements The information covered is classified according to its source.

4.5.1. FTS - Tonen Corp., H.S. Kliger & Associates, Structural Preservation Systems

Reference: [FTS-2]

The fatigue tests were performed on two bridge slabs taken from real structure after 26 years of service. One slab was strengthened with CFRP (Forca Tow Sheet FTS-C1-50) with two perpendicular layers. The strengthening was applied after performing 0.5×10^5 of initial cycling. A decrease in deflections of the strengthened slab by 20% was observed in the following series of cycling. Non-strengthened slab failed after 1.2×10^5 cycles of 15 ton load. The strengthened slab survived over 5×10^5 cycles of load without any breakage of carbon fiber sheet and no peeling-off. From the results of the tests it was concluded that CFRP is a very effective way to increase the fatigue endurance.

4.5.2. OH - Ohio Department of Transportation, Wright Patterson Air Force Base

Reference: [OH-3]

Tests were carried out on RC beams with external CFRP reinforcement. A commercial graphite/epoxy system (AS4C/1919 from Hercules Inc.) was used in the form of 3-ply prepreg. Fatigue damage was measured using two non-destructive methods: pulse velocity and natural frequency. Non-reversed fatigue load of sine wave form was applied up to 2 million cycles. Four control tests were tested statically for comparison. Beams which did not failed due to fatigue were tested statically to failure. The static flexural strength of control and fatigued beams were approximately the same. The endurance limit of beams reinforced with CFRP was greater than 250% of static flexural strength, which proved excellent fatigue properties of CFRP strengthening.

4.5.3. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-2]

Chajes tested 15 block specimens to evaluate their fatigue resistance of the composite material-concrete interfaces. The results indicated that fatigue resistance of the bond is a function of both the stress range and the maximum level of stress. Four of the five specimens cycled at a stress range of 25 % of the bond strength reached 2 million cycles without failing, and the unbroken specimens proved to be undamaged when tested statically. The three specimens cycled at a 45 % stress range and having a maximum stress of 55 % of ultimate reached, on average, 1.2 million cycles. The two specimen cycled at a 45 % stress range and having a maximum stress of 75 % reached, on average, only 187,000 cycles.

4.5.4. SOA - State of the Art Reports & Review Documents

Reference: [SOA-1]

The time -dependent behavior of concrete beams strengthened with FRP plates was studied by Kaiser (1989), Duering (1993), and Plevris and Triantafillou (1993). In a series of tests, failure under fatigue loading was always initiated by rupture of the tensile reinforcing bars. This resulted in transfer of stresses from the reinforcing bars to the CFRP, which eventually failed as well. Hence, the flexural capacity of the members was controlled by the strength of steel under repeated failure. Creep experiments were performed to determine the effect of CFRP on the behavior of the strengthened beams. It was concluded that the composite sheet can be modeled as a creep-free element perfectly bonded to the concrete.

Where the loading was repeated for 10 million cycles, carbon-epoxy composites have better fatigue strength than steel, while the fatigue strength of glass composites is lower than steel at a low stress ratio. (Schwarz 1992)

4.5.5. EMPA - Swiss Federal Laboratories for Materials Testing and Research

Reference: [EMPA-11] [EMPA-1]

CFRP laminates exhibit excellent fatigue behavior. Through the bonding of the CFRP laminates the inner reinforcement is relieved. Tests with very high vibration amplitudes yielded excellent results over more than 10 million load cycles.

Deuring performed fatigue tests on a strengthened beam with CFRP laminate with a span of 6m under realistic loading ranging from 125.8 kN to 283.4 kN, which is corresponding to 131 MPa-262 MPa for the rebars and 102 MPa-210 MPa for the laminate. After 10.7 million cycles the tests were continued in an environmental condition where the temperature and relative humidity are 40° C and 95 %, respectively. After a total of 12 million cycles the first steel reinforcement failed due to fretting fatigue. The joint between the CFRP laminate and the concrete did not present any severe strain fatigue. After 14.09 million cycles the second reinforcing bar failed due to fretting fatigue. After the failure of the third reinforcing rod, the CFRP laminate was sheared from the concrete.

Kaiser investigated a 2 m span beam strengthened with CFRP laminate under exaggerated fatigue loading ranging from 1 kN to 19 kN., which is corresponding to 12 MPa-407 MPa for the rebars and 11 MPa-205 MPa for the laminate. After 480,000 the first fatigue failure occurred in one of the two reinforcing rods in the tension zone; after 560,000 cycles the second reinforcing rod failed; after 750,000 cycles the first damage to the composite appeared in the form of fracture of individual rovings of the laminate; after 805,000 cycles the composite finally failed.

4.5.6. FTS - Tonen Corp., H.S. Kliger & Associates, Structural Preservation Systems

Reference: [FTS-6]

The 26 years old bridge slab strengthened with CFRP sheet was subjected to $0.5*10^5$ cycles under a 10 ton load level and $5*10^5$ cycles under a 15 ton load level. The strengthened slab did not break down after the test, while the unstrengthened slab did break down at $1.2*10^5$ cycles under the 15 ton load level.

4.5.7. XX - Other Research Institutions

Reference: [XX-26]

Under tension-tension fatigue load cycling between 10 % and 30 % of the tensile strength, both CFRP grid can endure 4 million cycles. After enduring 4 million cycles, the grid retained 93 % of its original tensile strength.

Reference: [XX-28]

Fatigue strength of CFRP rods was higher than that of GFRP and AFRP rods and the CFRP rods held more than 4 million fatigue cycles in the case that the maximum stress was less than 87.5 % of the mean tensile strength independent of the amplitude (Uomato, RILEM 95). This confirms the fact that Carbon fiber composites exhibit an excellent fatigue resistance.

4.6. Bond Properties

The interface bond between the FRP laminate and the concrete surface is one of the critical factors controlling the performance of the FRP plates strengthening systems. A number of studies investigating the flexural or shear behavior, incorporated also the issue of bond as a factor controlling the failure mode of the strengthened element.

In the following sections experimental studies aimed at investigating bond are reviewed.

4.6.1. FTS - Tonen Corp., H.S. Kliger & Associates, Structural Preservation Systems

Reference: [FTS-5]

Tests were performed using the axially loaded prism-shaped specimens to investigate the bonding strength of CFRP. Both high tensile (HT) and high modulus (HM) carbon fiber sheets were used. A total of 9 specimens was tested. The adhesive surface was prepared using the water jet and sander. In most samples the failure took place in the interface layer of concrete. Surface treated by water jet showed two times higher bonding strength than surface prepared by ordinary sander. The bonding strength increased also when the number of lamination layers was higher.

4.7. Analytical Models

Successfully use of FRP strengthening systems require development of proper analytical models. Existing models focus on flexural and shear capacity of concrete elements, particularly beam sections. The majority of models are based on the theory of compatibility of deformations and equilibrium of forces. One of the issues is the selection of the appropriate sequence of failures of the constituent materials. Brittle failure modes of the FRP materials should be avoided.

The following passages summarize the analytical models of FRP strengthening systems. The information covered is classified according to its source.

4.7.1. AZ -University of Arizona

Reference: [AZ-4]

Saadatmanesh, H and Ehsani, M.R presented analytical models based on the compatibility of deformations and equilibrium of forces to predict the stresses and deformations in concrete beams strengthened with Glass Fiber Reinforced Plastic (GFRP) plates epoxy-bonded to the tension face of the beams. It also investigate the effects of design variables such as steel reinforcement ratio, concrete compressive strength, plate area, and plate stiffness on the yield and ultimate moments of upgraded beams. Equations for calculations of strains and stresses in the FRP plate, steel rebar, and concrete, as well as the curvature at midspan, are calculated using an incremental deformation technique. Based on this study, it was found that the analytical models based on the compatibility of deformations and equilibrium of forces presented reasonably approximate the behavior of concrete beams externally reinforced with epoxy-bonded fiber composite plates, when a tough epoxy is used. Bonding a composite plate to a concrete beam can increase the stiffness, yield moment and flexural strength of the beam. It also reduces the curvature at failure. This method of strengthening is particularly effective for beams with a relatively low steel reinforcement ratio. Increase of the compressive strength of the concrete do not appreciably increase the ultimate moment of the section. Failure can be reach as a result of rupture of the plate, crushing of concrete in compression, or failure of the concrete layer between the plate and the reinforcing bars.

4.7.2. WVU - West Virginia University, Constructed Facilities Center

Reference: [WVU-1]

In the paper, the moment capacity increase due to longitudinal carbon tow sheet wrapping is discussed in terms of analytical modeling. The discussion is limited to failure modes after tension steel yielding of the wrapped concrete beam. Once the steel reinforcement yields, either the concrete can fail in compression or CFRP sheet can rupture due to tension. Other modes such as debonding, effect of creep, slip and aging of materials are not considered in the paper. The linear stress-strain curve was assumed for CFRP unidirectional sheet up to failure. The balanced, compressive and tensile types of failure are discussed and compared, depending on the section design. The analytical formulas describing each failure case are presented. The design procedure and flowchart are developed for the beam strengthened. The examples of design for different failure modes of beams wrapped with CFRP (Tonen Forca Tow) are included.

4.7.3. XX - Other Research Institutions

Reference: [XX-8]

The paper presents a numerical model for estimating the forces, moments, stresses, strains and deflections for any load level, of normal, damaged or repaired reinforced concrete beams. The model incorporates the stress-strain relationships for constituent materials. Creep and shrinkage can be also taken into account. The model was tested on a number of experimental data and proved its adequacy. Different material and geometrical properties can be applied in the computations.

Reference: [XX-19]

The paper presents the study of a balanced RC section strengthened with CFRP. Balanced conditions of RC section without and with FRP reinforcement are compared. The issues of Ductility and deformability are addressed. The recommendations for the design procedures are given.

Reference: [XX-20]

The reliability study of concrete structures in flexure having external FRP reinforcement is presented. The first part of the study concentrates on statistical character of design variables (geometrical and material properties) and their effect on material properties. The second part presents the methodology to establish the strength reduction factors for the given probability of failure. Third part evaluates the effect of each design variable on the reliability of the system.

Reference: [XX-38]

The paper presents the theoretical models for concrete beams prestressed with glued on FRP sheets. The models allow to estimate the maximum achievable prestress level so that the system will not fail near anchorage zones. Account is taken for the failure of either the adhesive layer or the beam material, depending on which of the two possesses lower shear strength. It is concluded that the method efficiency can be improved by increasing

the thickness of adhesive layer. The maximum achievable prestress level increases as both the adhesive thickness and the composite sheet cross section area fraction increase.
For FRP prestressed concrete beams additional anchoring mechanical devices may be necessary.
The presented model is valid for short-term behavior of FRP reinforced elements. The long-term viscoelastic behavior (creep and relaxation) has to be investigated,

4.7.4. SOA - State of the Art Reports & Review Documents

Reference: [SOA-1]

Ritchie (1991) tested a series of concrete beams strengthened with GFRP, CFRP, and AFRP and developed an analytical method based on strain compatibility to predict the strength and stiffness of the plated beams.

Triantafillou and Plevris (1990,1992) used the strain compatibility method and an analytical model for the FRP peeling -off mechanism based on the shearing dowel actions of both the steel reinforcement and the FRP plate to study the short-term flexural behavior of reinforced concrete beams strengthened with FRP laminates. The analytical results of failure mechanisms and corresponding loads were validated through a series of experiments employing thin CFRP sheets.

Plevris and Triantafillou (1993) developed an analytical model for predict the creep and shrinkage behavior of concrete beams strengthened with various types of FRP plates. It was concluded that:

- CFRP and GFRP affect the long-term response of strengthened elements.
- Increasing the area of these materials decreases the creep strains without affecting the time-dependent curvature, the tensile steel reinforcement stress, and the stress in the composite material.
- Increasing the area of FRP in general, tends to restrain the reduction of stress in the concrete compressive zone.

Plevris (1993) analyzed the flexure behavior of concrete beams strengthened with CFRP sheets. The concrete strength, CFRP failure strain, and CFRP area fraction were found to be the most influential on the variability of member strength. A reliability-based design procedure was also developed: Two strength reduction factors were derived to achieve a reliability index of about three over a broad spectrum of design conditions. It was concluded that the concrete, steel and ratio of live to dead load all have important effects on the reliability against flexural failure.

4.8. Durability

Civil Engineering structures are permanently exposed to aggressive agents. The use of FRP composite materials sheets has the advantage of corrosion resistance as compared to the use of steel plates. Other factors such as freeze-thaw conditions, fire resistance, ultraviolet radiation, chemical aggression and low temperatures require further investigation.

This new technology lacks long term infield experience about durability performance of the FRP sheets. Therefore experimental tests of these factors are needed.

The following sections summarize the experimental studies of durability of FRP strengthened concrete elements. The information covered is classified according to the durability factor tested and its source.

4.8.1. CA-CALTRANS California Department of Transportation

References: [CA-7,8,9,10]

Several composite overwrap systems have been proposed to the State of California, Department of Transportation (Caltrans) as alternative column casings for seismic retrofit. Caltrans prepared the document, "Prequalification Requirements for Alternative Column Casings for Seismic Retrofit", to provide a detailed listing of the requirements for prequalification of materials and process for composite column casing. These requirements include durability testing to demonstrate the ability of the proposed composite material systems to withstand a variety of climatic and unnatural exposure conditions. Environmental exposure include: 100% humidity, salt water, diesel fuel, cyclic ultraviolet light/water condensation, alkali solution, elevated temperature (140 F), and cyclic freezing and thawing. All testing has been performed by The Aerospace Corporation for Caltrans.

Flat laminates of each candidate composite system was subjected to these environmental exposure for various times or numbers of cycles as shown in Table 4.2. A partial test results summary is given in Table 4.3

Table 4.2 Environmental Durability Test Matrix for Composite Laminates

Environmental durability test	Relevant specification	Test conditions	Test duration
Water resistance	ASTM D 2247 ASTM E 104	100% Humidity at 100±2_F	1000, 3000, & 10000 hours
Salt water resistance	ASTM D 1141 ASTM C 581	Immersion at 73±3_F	1000, 3000, & 10000 hours
Alkali resistance	ASTM C 581	Immersion in $Ca(OH)_2$ at pH=9.5 & 73±3_F	1000, 3000, & 10000 hours
Dry heat resistance	ASTM D 3045	140±5_F	1000 & 3000 hours
Fuel resistance	ASTM C 581	Immersion at 73±3_F	4 hours
Ultraviolet resistance	ASTM G 53	Cycle between UV at 140_F & condensate at 104_F	4 hours per condition, 100 cycles
Freeze-Thaw resistance	None	Cycle between 100% humidity at 100_F & freezer at 0_F	24 hours per condition, 20 cycles

Table 4.3 Environmental Durability Test Results after 3000 Hour Exposure

Environmental exposure	Normalized Young's modulus				Normalized tensile strength				Failure strain (%)				Normalized short beam shear strength			
	C1	C2	C3	C4	C1	C2	C3	C4	C1	C2	C3	C4	C1	C2	C3	C4
Control	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.65	1.50	1.40	1.50	1.00	1.00	1.00	1.00
Humidity	1.08	1.03	1.04	1.00	0.95	0.91	1.04	0.98	1.50	1.35	1.40	1.50	0.68	1.13	1.11	0.83
Salt	1.02	1.02	1.05	1.01	0.89	0.92	0.94	0.96	1.50	1.30	1.25	1.45	0.72	1.03	1.02	0.88
pH 9.5	0.99	1.03	0.99	1.00	0.95	1.03	0.93	0.97	1.60	1.50	1.35	1.50	0.72	1.17	1.06	0.83
140 F	0.97	1.01	1.00	0.98	1.04	1.06	0.99	0.97	1.75	1.50	1.35	1.50	1.18	1.12	1.11	1.01
Freeze/Thaw	1.00	1.07	1.04	1.01	0.92	1.05	0.99	0.94	1.55	1.45	1.35	1.45	0.77	1.12	0.94	0.88
Ultraviolet	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Diesel fuel	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Note: C1, C2, C3, and C4 are different carbon fiber composite systems.
C4 = Tonen Carbon sheet

4.9. Low Temperature

The difference in the thermal expansion coefficients of concrete and of CFRP sheet results in stresses at the interface with changing temperatures. Indeed concrete and CFRP have significantly different coefficients of thermal expansion, normally:

Thermal expansion coefficient of concrete: $10 * 10^{-6} / °C$
Thermal expansion coefficient of CFRP: $< 1 * 10^{-6} / °C$

4.10. Freeze-Thaw

The rehabilitation technique of external reinforcement of concrete elements by the use of CFRP sheets can not be effectively utilized in cold regions without investigating the durability of this new material in low temperatures environments.

Table 4.4. summarizes the most important points extracted from the literature review of freeze-thaw durability of concrete elements strengthened by CFRP sheets.

Table 4.4 Test Parameters Found on Literature Review about Freeze-Thaw (F.T.) Test

Reference	No. specimens	No. cycles	Tests Procedures	Type of FRP	Variable	Comments
DE-2	60 beams 38.1x 28.6x330mm Reinf Ø =2.38mm Adhes. = Sikadur 32 Aggregate size = 3.175 mm	50, 100	Epoxy-concrete compatibility ASTM C884-87 Freeze-thaw (F.T.) ASTM C672-84 Flexural test (4 pts) after environmental cycling	Aramid, E-glass, Graphite (Carbon)	type of FRP: Aramid, E-glass, Graphite (Carbon)	■ Graphite has higher env. durability than Aramid and E-glass. ■ For CFRP 21% decrease in strength compared with 17% of unwrapped beams. ■ Wet/dry cycles more critical than Freeze-thaw test. ■ Change in failure mode due to environmental exposure. ■ Partial debonding prior to failure
DE-3	Field application Foulk Road bridge	(1 year)	6 beams	CFRP	- Performance during its first year. - Evaluation of bond characteristics. - Durability of joints	■ All of the single layer applications of CFRP were well bonded (5 beams) ■ 1 beam with 2 layers of the highest strength sheet was not well bonded. Assumed due to insufficient resin saturation during the bonding process.
XX-9	2 coupons 25.4 x 9.53 x 304.8 mm	300	Cut edges coated with epoxy 2%water solut. Temp = 0-40 F F.T. (ASTM C666) Flexure (ASTM D790)	Fiber glass composite	Edge coated-uncoated Salt water	■ Lost 20-30% flexural strength, rigidity and toughness ■ Loss due to only salt water exposure =5-10%
XX-25	FRP rods embedded in a concrete cube of 10 cm length	300	bond test CP110 ASTM and RILEM	GFRP, CFRP and Vynylon FRP bar	Bond strength	■ No great influence of the bond strength due to F. T. Cycles.

Reference	No. specimens	No. cycles	Tests Procedures	Type of FRP	Variable	Comments
XX-26	N.A.	N.A.	N.A.	CGFRP grid (74% glass, 26 % Carbon)	Range T	■ Increase ultimate tensile strength by 11% and 2.5% mod. Elasticity for a fall in Temp. of -30C ■ Decrease of 9.5% and 2.5% respect. for increase from room T to 50C.
XX-22	N.A.	N.A.	N.A.	N.A.	Combine env. Expos. Crack propagation	■ The freezer effect had little effect on crack propagation
CAN-1	42 columns, 15 were subjected to FT test 152 x 305 mm	200	Cycling = -18C to +20C	CFRP wrap	% of Reinf. # of CFRP layers Env. Conditions: F.T. test Low Temperature water	■ CFRP wrapped concrete columns exposed to F. T. cycling showed a significant increase in strength (3 times) compared to unwrapped columns exposed at the same cond. ■ A second layer of CFRP provides an increase of 15% in strength ■ The wrapped columns subjected to F.T. cycling failed in a more catastrophic manner than those at room Temperature.
CAN-2	18 beams	N.A.	40C one week, -23C one week for 2 months Flexural test (3pt. Bending)	N.A.	Accelerated env. Exposure	■ 7% Reduction in strength for beams subjected to this hot-cold cycles. ■ Results from hot-cold cycles are quite close to those of long term exposure. ■ The effect of Temperature is more important than humidity in term of reducing the bonding strength.
CAN-4	12 beams 102 x 152 x 1220 mm As = 4 # stirrups	50	F.T. cycle -18C to +20C (cold room overnight to water bath)	CFRP	- # of CFRP sheet (0,1) - Orientation of sheet (long vs. Transv.) - Freeze-thaw vs. Room Temperature	■ No decrease in ultimate strength due to F.T. cycles ■ Strengthening with CFRP improves strength and ductility. ■ No difference in failure mode when compared with control beams. ■ F.T. cycling affect cracking behavior but does not affect ultimate behavior

N.A. = Information not available

In the following sections experimental studies aimed at investigating freeze-thaw durability of concrete elements strengthened by CFRP sheets are reviewed.

4.10.1. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-2]

At the University of Delaware, tests were performed in order to investigate the environmental durability of the composite fiber materials: Aramid, E-glass and graphite (carbon) fibers. The environmental durability studies involved subjecting the 60 small scale beam specimens to aggressive environments (the beams were exposed to repeated wet/dry and freeze/thaw cycles while sitting in a solution of calcium chloride.) Exposure times were varied and the flexural capacity of the exposed beam were compared with those not exposed. Once the beams completed the environmental cycling they were loaded to failure in four point bending. The results of the test indicate that the beams reinforced with the graphite fabric have greater environmental durability than those with Aramid or E-glass. After being subjected to environmental conditions, only graphite reinforced beams maintained nearly all of their strength advantage over the unwrapped beams. Graphite fabric proved the most suitable in applications involving environmental aggression. While both the Aramid and E-glass reinforced beams showed a 36 % decrease in strength due to 100 wet/dry cycles, the graphite reinforced beams dropped in strength by only 19 % (as compared to 12 % drop in strength of the unstrengthened beams). For freeze-thaw test,100 cycles, a decrease of 21% on the beam strength was reported (compare with 17% of unwrapped.) Of the two conditions the wet/dry cycling led to greater degradation. The tests also revealed that the environmental exposure can lead to changed mode of failure compared to non-exposed specimens. Partial debonding was experienced prior to failure.

Reference: [DE-4]

By Finch et al., bond tests using single-lap shear test method are being conducted on specimens(25.4 mm-wide graphite/epoxy plate bonded to a concrete block) which has been (1) immersed in fresh and saline water, (2) exposed to winter weather, and (3) subjected to repeated wetting and drying cycles.

4.10.2. Other Research On Freeze-Thaw

Reference: [XX-9]

Cycles of freezing and thawing temperatures will probably magnify the effects of water absorption. The expansion of the freezing water causes further delamination and interfacial failure. Two commercially available fiber glass composite coupons were placed in a 2 % salt water solution and subjected to 300 cycles of freezing and thawing, with the temperature ranging -17.8° C (0° F) and 4.4° C (40° F). Results indicates significant loss(20-30 %) in flexural strength, rigidity, and toughness. For only salt water, the percentage reduction is 5-10 %.

Reference: [XX-25]

Glass, Vynylon and carbon FRP bar is not influenced to the bond strength so much by 300 cycles of freezing and thawing. Aramid FRP (braided and coiled types) reduce bond strength gradually with process of freezing and thawing (by 50 % at 600 cycles).

Reference: [XX-26]

The ultimate tensile strength of Carbon-Glass Fiber Reinforced Plastic (CGFRP) grid (74 % glass+26 % carbon) increases by 11 % and modulus of elasticity by 2.5 % when the temperature falls to -30° C. On the other hand, tensile strength and modulus of elasticity decrease by 9.5 % and 2.5 %, respectively, when the temperature rises to 50° C from the room temperature.

Reference: [X-22]

Bourban et al. studied the combined effects of stress and environmental exposure on the steel-composite joints with three epoxies using the wedge-crack extension test. The crack propagation was affected by hot water, salt water, room temperature water, and freezer-thaw in the order of important environmental effect. The freezer itself had little effect on crack propagation.

Reference: [XX-27]

Shulley et. al investigated the durability of five different types of reinforcing fibers(3 carbon and 2 glass) bonded on steel using the wedge-crack extension test. Different types of fibers had different durabilities against different environments. The crack growth was affected by hot water(65° C) , sea water, aqueous environment, and freeze-thaw (-18° C~20° C)in the order of important environmental effect. A sub-zero environment(-18° C) had little effect on crack growth (a slightly positive effects for some fibers).

Reference: [XX-29]

Based on results of a survey of 40 to 75 year old, non-entrained concrete dams, a new one-sided freezing test is proposed. In this survey, most of the deterioration was found in areas away from the direct exposure to water, but subject to many cycles of freezing in air. The unfrozen side is exposed to water and the freezing side is exposed to air, simulating a filed exposure

Reference: [XX-30]

Stresses developed in composites due to changes in Temperature and Relative Humidity are due largely to mis-matches in the different moisture and thermal coefficients of expansion of the materials. In this paper, a general model is developed directly related to the temperature and humidity related restrained response of materials. This information is useful in developing predicting numerical models for the behavior of composites structures.

4.10.3. EMPA - Swiss Federal Laboratories for Materials Testing and Research

Reference: [EMPA-10]

When a change of temperature takes place the difference in the coefficient of thermal expansion of concrete and the composites result in thermal stresses at the joints between the two components. After 100 frost cycles ranging from 20°C to -25°C, no negative influence on loading capacity of the three post-strengthened beams was found.

Reference: [EMPA-1]

Meier reported that when a change of temperature takes place, the differences in the coefficients of thermal expansion of concrete and the carbon fiber composites resulted in thermal stresses at the joints between the two components. After 100 frost cycles ranging from +20 °C to -25°C, no negative influence on the loading capacity of three post-strengthened beams tested was found.

4.10.4. CAN - Canadian Institutes and Universities

Reference: [CAN-01]

This paper presents the results of the performance in cold weather of circular concrete columns strengthened with CFRP wraps. 42 circular plain and reinforced concrete columns were tested. Test variables included percentage of reinforcement (0%. 1%, 2.3%) number of CFRP layers (0, 1, 2) and environmental exposure conditions(room Temperature, Freeze-thaw cycles -18 to 20C) and under water). 15 columns were submerged in water for 200 freeze/thaw cycles. After completion of the cold climate exposure, the columns were tested for axial strength and load versus axial strain plots were obtained. Based on the results of the experimental program, CFRP sheets proved to be feasible in strengthening columns in cold regions.

Reference: [CAN-2]

Beams of dimensions 51 x 76 x 279 mm, were externally reinforced with a Carbon/epoxy composite sheet and tested under flexure after being submitted to different environments. Unidirectional graphite/epoxy composite sheets were made by autoclave-vacuum molding using Newport NCT-301 composite prepeg. Thickness varies from 0.33 mm for 3 layers to 6 mm for 45 layers. Ciba-Geigy's structure epoxy adhesives AW106 and Rp1700-1 were used for the bonding procedure

Results showed that accelerated environmental exposure by water immersion for 60 days had slight positive effect on the load bearing capacity. Whereas hot-cold cycles for 60 days and long term outdoor exposure up to 28 months reduced the load bearing capacity for about 7%. The effect of temperature was more important than humidity. Hot-cold cycle is an effective method for accelerated test.

Reference: [CAN-3]

This paper reviews the existing information on the low temperature response of reinforced concrete members strengthened with FRP sheets. It reviews the material behavior of concrete, steel, and FRP at low temperatures, and discusses the observed behavior of reinforced concrete beams, with and without FRP strengthening when subjected to low temperatures.

Experiments on tensile loading of unidirectional FRP at low temperature (Dutta, 1990) have shown that the longitudinal strength of these composites drops at low temperatures. It is generally believed that, in unidirectional FRP with a high fiber volume fraction, tensile is primarily governed by the fiber properties.
To investigate the effect of freeze-thaw cycles, 6 test beams were subjected to 100 freeze/thaw cycles of 20C to -25C before being tested to failure in four point bending at room temperature (Kaiser, 1989). Half of these beams were cracked prior to adhesion of the laminate. During temperature cycling, the frozen beams were thawed by flooding the freezers with water at a temperature of approximately 20C. It was expected that water would enter into cracks and expand with subsequent freezing, resulting in peeling of the laminate. All frozen beams were brought to room temperature before being tested. A comparison of the breaking loads of the frozen beams with the breaking loads of the control beams showed no negative influence on the ultimate load capacity.
Concrete beams strengthened with FRP sheets may increase in strength when subjected to short-term exposure to low temperatures. Long-term exposure of such members must be investigated to determine the effects of creep and aging of the materials.

Reference: [CAN-4]

This paper presents the results of an investigation into the effects of Freeze- thaw cycling on the flexural and shear behavior of beams post-strengthened with CFRP sheets. 12 rectangular beams with different steel and CFRP reinforcement configuration were subjected half to 50 freeze-thaw cycles and half were kept in room temperature. All the beams were finally subject to 4 point flexural test. Researchers concluded that CFRP sheets are effective in strengthening flexural members exposed to freeze-thaw cycling. CFRP sheets can be used as external shear and/or flexural reinforcement. Strengthening concrete beams with CFRP sheets improve strength and ductility. Failure mode observed were:

- bond peeling of CFRP sheet (BF).
- rupture of CFRP fibers (FD)

No difference in failure mode between beams subjected to freeze-thaw and those at room Temperature. Freeze-thaw slightly affects cracking behavior of beams, but does not affect ultimate behavior. Finally, the theoretical predictions compare well with test results.

4.11. Chemical Aggression - Chloride and Salt Water

Excessive absorption of water in composites could result in loss of strength and stiffness. Water absorption produces changes in resin properties and could cause swelling, warping, and delamination in composites [SOA-1]. However, there are resins are formulated to be moisture-resistant and may be used when a structure is expected to be wet at all times.

The following sections summarize the experimental studies of durability of FRP strengthened concrete elements under weathering conditions and salt water effect.

4.11.1. MRK - Mitsubishi (Replark), Craig Ballinger & Associates

Reference: [MRK-3]

In weathering test of CFRP sheet, no reduction was observed in tensile and bond strength for 2000 hours of accelerated exposure period equals to about 7 years. The durability of the CFRP rod against salt water was checked in salt water for 1 year. No reduction of tensile strength was observed.

4.11.2. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-02]

Chajes et al. investigated the environmental durability of the composite fiber materials: Aramid, E-glass and graphite (carbon) fibers. The environmental durability studies involved subjecting the 60 small scale beam specimens to aggressive environments (the beams were exposed to repeated wet/dry and freeze/thaw cycles while sitting in a solution of calcium chloride.) The results indicate that the wet/dry cycling had a slightly more severe effect on the beams than did the freezing/thawing cycling. (Refer to section 4.10 Freeze-Thaw)

Reference: [DE-4]

By Finch et al., bond tests using single-lap shear test method are being conducted. (Refer to freezing and thawing).

4.11.3. XX - Other Research Institutions

Reference: [XX-9]

Two commercially available fiber glass composite coupons were exposed to 300 cycles of 2 % salt water. The percentage reduction in flexural strength is 3-7 %.

Reference: [XX-22]

Bourban and Shulley conducted their wedge-crack extension tests on the interface between steel and composites with different epoxies and different fibers, respectively. (Refer to freezing and thawing)

4.11.4. HF- HEXCEL FYFE Co.

Reference: [HF-1]

Fyfe et al. evaluated the durability of Tyfo™ S System(glass fiber) which is used to repair column. The test results show that salt water exposure did not effect the tensile strength of the panels.

4.11.5. FL - Florida DOT

Reference: [FL-1]

Sen et al. investigated the durability of S-2 glass/epoxy pretensioned beams subjected to cycles of wetting and drying with 15 % salt water solution to simulate tidal effects. The tests indicate a complete loss in the effectiveness of the fiberglass strands after 6 months for the precracked beams and 15 months for the uncracked beams.

4.12. Ultraviolet Light

Composites can be damaged by the ultraviolet rays present in sunlight. These rays cause chemical reactions in a polymer matrix, which can lead to degradation of properties. The problem can be solved with the introduction of appropriate additives to the resin[SOA-1.]

4.12.1. HF- HEXCEL FYFE Co.

Reference: [HF-1]

Fyfe studied weathermeter aging of Tyfo™ S System(glass fiber) exposed to ultraviolet and condensate. The test data show that the failure loads of the panels did not change significantly. He also studied thermal aging of the laminate. The results indicate that 140° F thermal aging had no negative effect.

4.13. Fire Resistance

Due to the different properties of the components of a laminate, the effect of temperature is more severe on the resin than on the fiber [SOA-1]. An adequate value of fire resistance of the FRP strengthened concrete elements is necessary to meet safety conditions provides by construction codes.

The following sections summarize the experimental studies of fire resistance of FRP strengthened concrete elements.

4.13.1. EMPA - Swiss Federal Laboratories for Materials Testing and Research

Reference: [EMPA-1][EMPA-10]

In 1994, by the EMPA, the 6 m beams strengthened with steel and CFRP sheets under 4 point loading were heated with a temperature of 925 k after 1 hour. After 8 minutes the steel plate came away from the beam strengthened with steel plate. During the test the fibers started to burn at the surface of the laminates of the 4 strengthened beams, thus causing a slow decreases in stiffness. The CFRP composites finally became unbonded from the beams after one hour. The main reason for the superior behavior of the CFRP composites compared with that of the steel plates was their low conductivity in the lateral direction.

4.13.2. MRK - Mitsubishi (Replark), Craig Ballinger & Associates

Reference: [MRK-3]

For CFRP sheet(with epoxy resin), the maximum fire resistant temperature is 260° C for 2 hours based on the fact that no strength reduction was observed after returning to ambient temperature.

4.13.3. XX - Other Research Institutions

Reference: [XX-23]

The tension test under heating for H8 (glass+carbon) grid shaped FRP, the heated strength decreases gradually from room temperature to 100° C, and remain about 60 % of the unheated tensile strength at around 100 to 250° C (20 minutes at 250° C). After cooling to room temperature, the heated strength return to its unheated tensile strength.

4.14. Field Applications

FRP strengthening systems have been used successfully in a number of field applications. Strengthening of flexural elements is among the most common usage, because the advantages of this new technique for the strengthening of flexural elements have been proved in numerous field applications in the United States and abroad.

The following sections summarize the field application of FRP strengthening systems in concrete structures. The information covered is classified according to its source.

4.14.1. FTS - Tonen Corp., H.S. Kliger & Associates, Structural Preservation Systems

Reference: [FTS-1]

A highway bridge having RC slab mounted on steel girders was strengthened with Forca Tow Sheet FTS-C1-30. The bridge after over 20 years of service exhibited many cracks in deck in both longitudinal and transverse directions. The fiber was applied as one longitudinal and one transverse layer with total area of 164 m^2. The application of strengthening took approximately two weeks. Traffic was not interrupted. The load tests were carried out prior and after the rehabilitation and the measurements readouts compared. The test were done at low (5 km/h) and normal (40 km/h) speeds of loading trucks. The deflections of the slab itself decreased by 15 to 20 percent. The strains of the main reinforcement were reduced by approximately 30 to 40 percent. Similarly, the strains in distribution bars decreased by approximately 20 to 40 percent.

References: [FTS-2] [FTS-3]

The CFRP Forca Tow Sheet was applied on Shirota bridge in Kyushu Province (Japan). Due to relaxing of the allowable loads (from 20 to 25 Tons) the stresses in main and distribution bars of the bridge slab exceeded the allowable limit of 1200 kg/cm^2 (17 ksi). The CFRP strengthening was designed to reduce the stresses below the allowable value. After the repair, the measurements were done using the strain gauges mounted at main and distribution bars. The measurements confirmed the substantial decrease of deflections and stresses by more than 30%. To confirm the long term reinforcing effect the strain gauges were monitored over a one year period. The results clearly indicate that the reinforcing effect is still effective after one year.

4.14.2. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-3]

The externally applied CFRP sheets were bonded to the beams of Foulk Road Bridge #26, Wilmington, Delaware. The superstructure consists of 24 simple-span adjacent box-beams, each 16.4 m long. The beam cross-section is 0.9m wide and 0.7mm deep, and has the wall thickness of approximately 130 mm. Six out of the 24 beams were retrofitted. The application was performed during a one-week period in October of 1994, taking a 5 men crew working 5-8 hours daily. The methods of monitoring the structure (both rehabilitated and unrehabilitated beams) relayed on periodic visual inspection, ongoing monitoring of ambient conditions, bond performance and crack growth. The initial inspection was performed immediately after application of CFRP. A sophisticated instrumentation was installed on the bridge to monitor ambient conditions. To date the Foulk Road Bridge rehabilitation performed well. No signs of deterioration was noticed in the applied composite. The unrehabilitated beams showed noticeable progress in cracking compared to rehabilitated ones.

4.14.3. Other Applications in North America

- Column wrapping projects, California: As a part of its general seismic upgrading program, the California Department of Transportation (Caltrans) placed confining jackets around bridge columns using fiberglass mat.
- Column wrapping projects, Reno, Nevada: In 1993, the Nevada Department of Transportation wrapped 96 0.3 m (3 ft) diameter columns with a proprietary FRP wrapping system.

- Strengthening of walls, Glendale, California: Fiber composite fabrics can be epoxy-bonded to the surfaces of masonry or concrete walls to increase the strength of these elements.
- Foulk Road bridge, Wilmington, Delaware: Carbon fiber Forca TM tow sheets were used on this 16.5 m (54 ft) long, simple span, prestressed, precast box beam structure that exhibited cracking indicative of the lack of transverse reinforcement. The bridge's superstructure was composed of 24 prestressed box beams placed adjacent to each other. For demonstration purposes, six of the beams were retrofitted. One ply of unidirectional CFRP sheet, with the fibers running transverse to the beam, was used on four beams. Two other beams used higher modulus, higher weight fabric, with on of those beams fitted with two plies rather than a single ply

4.14.4. Other Applications in Japan

- Wrapping projects: Forca Tow Sheet has been used extensively in Japan over 200 projects including tunnels, chimneys, side walls, and slabs.
- Carbon fiber sheet Replark ™ has been utilized as a strengthening system in Japan. Columns, road decks, bridge piers, bridge girders, retaining walls, slabs, floors and beams were repaired.

4.14.5. Other Applications in Europe

- Kattenbusch bridge (Meier 1987), an eleven-span post-tensioned concrete bridge consisting of two hollow box girders was strengthened with GFRP plates. The bridge was built with working joints at the points of contraflexure, where wide cracks appeared several years after construction. Additional reinforcement to control the crack width and to reduce the tendon stresses was provided by strengthening eight joints with steel plates and two joints with GFRP plates (30 mm thick, 150 mm wide, and 3200 mm long) per box girder. The plates were bonded to the top face of the bottom flange.
- Ibach bridge, Switzerland (Meier et al. 1992): Accidental damage to a prestressing tendon during maintenance work necessitated repair of this bridge in 1991. The bridge was strengthened with 5 m (16.5 ft) long CFRP laminates, two with a 150 by 1.75 mm (6 by 0.07 in.) cross sections and one with a 150 by 2.0 mm (6 by 0.08 in.) cross section, were applied to the bottom surface of the bridge. Approximately 14 lb. (6.2 kg) of CFRP were used in lieu of 385 lb. of steel.

CHAPTER 5
CONCLUSIONS

- Substantial amount of research done on FRP composites demonstrated the feasibility of the utilization of glued-on sheets as a strengthening technique for concrete structures.

- Carbon FRP composites seem, among different FRP laminates reviewed in this study, the most suitable for civil engineering applications. They possess excellent mechanical and durability properties.

- The best known CFRP systems currently used are produced by Tonen, Mitsubishi and Sika. Tonen and Mitsubishi products have similar characteristics. The Sika product differs in thickness and smaller choice of adhesives. Tonen *Forca Tow Sheet* is the most versatile product. The wide choice of different adhesives and primers makes it suitable for different conditions of applications.

- The simple application procedures developed, have made it possible to utilize CFRP sheets in field applications. However, the application of commercial FRP strengthening systems requires strict following of the specified procedures. So far, no standardized guidelines have been developed for this technique. Only by following the procedures recommended by each commercial product manufacturer the proper quality of strengthening can be achieved.

- The choice of the fiber strengthening system should in all cases be based on consideration of specifics of the application, such as:

  - purpose of strengthening;
  - the design of the structure or element;
  - conditions of application of FRP system (accessibility, temperature, humidity, degree of structural damage, surface preparation procedures etc.);
  - risks involved;
  - stress levels likely to occur in the retrofit system;
  - duration for which the repair/retrofit is being designed for.

- Durability issues should be addressed by a more intensive experimental program to ascertain the feasibility of this technique under particular environmental conditions.

- One of the key issues in successful application of CFRP composites is proper preparation of adhesive surfaces. The greater the level of damage (or contamination) of concrete cover, the more aggressive methods are necessary to remove the contaminated layer.

- A field test is recommended as the best way to corroborate findings performed in laboratory conditions.

- The high costs of the CFRP materials used in structural rehabilitation is fully compensated by the ease of application and low costs of workmanship.

- Not all issues have been fully explored. Further tests are necessary in order to identify the influence of different physical, mechanical and structural factors on performance of FRP laminates. In particular durability behavior of FRP composites requires further investigation.

CHAPTER 6
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FL - Florida DOT

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MI - Michigan Department of Transportation

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VHRC - Virginia highway Research Council

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TRRL - Transport and Road Laboratory. Dept. of the Environment. Dept. of Transport

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ASTM - American Standard Specifications

ASTM-1. ASTM C666-92. Standard Test method for Resistance of Concrete to Rapid Freezing and Thawing.

7.1. Glossary of Terms and Abbreviations

AFB	Air Force Base
DOT	Department of Transportation
FHWA	Federal Highway Administration
FTS	Forca Tow Sheet (Tonen Corporation)
MRK	Mitsubishi Replark
SCD	Sika (CarboDur)
FRP	Fiber Reinforced Plastic
GFRP	Glass FRP
CFRP	Carbon (graphite) FRP
AFRP	Aramid FRP

7.2. Contacts to Organizations Working on FRP

Table 7.1 Contacts To Organizations and Research Institutions

Institution	Department	Address	Contact person	Telephone/ Fax No. E-mail address	Comments on Research or Products	Comments on Contact
State and Federal Agencies						
MDOT Michigan DOT	Materials and Technology	P.O. Box 30049 8885 Ricks Road Lansing, Michigan 48909	Roger D. Till	(517) 322 5682 phone (517) 322 5203 fax tillr@state.mi.us		Project Coordinator
ODOT Ohio DOT	Bridge Bureau	25 South Front Str. Rm 516 Columbus, Ohio 43216	Mr. Steven Morton	(614) 466 4318	In the process of lab testing. Field Testing scheduled for spring. Possible visit to testing place.	Promised to send materials. Package received on Feb.7th.
DDOT Deleware DOT		Administration Center PO BOX 778 Dover, DE 19901	Mr. Chao Hu	(302) 739 4341	Performed research in cooperation with UoDE. CFRP from Tonen used.	Left message on Jan 22nd. Replied on Jan 24th. He gave contact to Prof. Michael Chajes (UoDE) who should have a report.
FDOT Florida DOT		605 Suwannee Street Tallahassee, FL 32399-0450	dr. Mohsen Shahawy	(904) 488 6179 phone (904) 488 6189 fax		Promised to send materials. Fax with address sent to him.
CDOT California DOT CALTRANS			Mrs. Beverly Mason	(916) 227 8256	Research and field applicactions in cooperation with University of California - San Diego and Hexcel Fyfe Coompany.	Contacted on March 24th following the conversation with Roger Till. Materials received through MDOT on April 9th.
CDOT California DOT CALTRANS		MS-9 Engineering Services Center 1801 30th St W Sacramento, CA 95816-5000	Mosen Soltan	(916) 227 8247	Person involved in CFRP in Caltrans.	Did not respond personally.
CDOT California DOT CALTRANS	New Tech. & Mat.; Transp. Lab.			(916) 654 9322 (916) 227 7000		Promised to respond and find proper contact person.
Air Force Wright Laboratories	Materials Directorate, Nonmetallic Materials Division	WL/XPT Building 45 2130 Eight Street Ste 1 WPAFB OH 45433-7562	Ms. Cindy Ingalls (secretary) Tech Transfer	(513) 255 2788 phone (513) 656 4572 fax ingallfs@wl.wpafb.af.mil	http://www.wl.wpafb.af.mil/stories/ss15_96.htm	Sent a fax with basic facts sheet. Gave contact to John Mistretta on (937) 255 9059
- || -	Materials Directorate, Technology Transfer Center	- || -		(513) 255 4689		Call. Refer to item 95-166.
- || -	- || -	- || -	John Mistretta Scott Thiebert (technical contact)	(937) 255 9059 J.M phone (513) 255 9018 S.T. phone	Worked with ODOT (Steven Morton). They have joint report.	Talked to Scott Thiebert. He was reluctant to help. Promised to send some materials.

US Corps of Engineers	Waterways Experiment Station	Visksburg	Edward O'Neil	(601) 634 3387 phone (601) 634 3242 fax	Did research on fiber prestressing tendons. Nothing on glued plates.	Contacted
US Corps of Engineers	Construction Engineering Research Laboratories	UASCERL ATTN: CECER-FL-M, P.O. Box 9005, Champaign, IL 61826-9005	Mr. Jonathon Trovillion	(217) 352 6511 (800) USA CERL or (800) 872 2375		
FHWA Federal Highway Administration	HNR-10 Structures Division	6300 Georgetown PK McLean, VA 22101-2296	Mr. Eric Munley	(703) 285 2438	He gave coordinates of Mitsubishi, Tonen, Sika. Two major techniques: • bonded plates • wet layout	Contacted on Jan 23. In the process of moving to new building.
Companies						
Mitsubishi Chemicals America Inc.	California Office	99 W. Tasman Drive, Suite 200, San Jose CA 95134-1712	Michihiko Sakamoto (Sales Manager Carbon Fiber)	(408) 232 6280 phone (408) 954 8494 fax	Mitsubishi Replark	Package sent to S.Y.Park. Info about adhesives faxed to S.Y.Park on March 6th.
Tonen Corporation		Place Side Bldg. 1-1-1, Hitotsubashi, Chiyoda-ku, Tokyo 100, Japan		03-3286-5104 phone 03-3286-5074 fax	Tonen Forca Tow Sheet	Materials received through US representatives.
Sika USA		201 Polito Avenue Lyndhurst, N.J. 07071	David White, P.E. Product Manager	(201) 933 8800 ext.269 (201) 933 6225 fax (800) 933 7452 toll free	Sika CarboDur - Carbon Fiber Reinforced Polymer.	Talked to David White. Promised Materials. Contacts: dr. William Spillers & Prof. Charles Dolan
Sika Corporation	US Office	2190 Gladstone Court Suite A Glendale Heights, IL 60139	Etien Frett Tech. Representative	(708) 924 7900 phone (708) 924 8508 fax (800) 933 SIKA - tech. service	Sika CarboDur - Carbon Fiber Reinforced Polymer.	Materials sent to S.Y.Park
Sika AG	Main Headquarters	P.O. Box 1300 CH-8048 Zurich/ Switzerland		01 436 40 40 phone 01 432 33 62 fax 822 254 sik ch - telex	Sika CarboDur - Carbon Fiber Reinforced Polymer.	Materials received through US representatives.
Amoco Polymers, Inc.		4500 McGinnis Ferry Road, Alpharetta, GA 30202-3914		1-800-222-2448		Contacted. Promised to send materials. No response.
Du Pont	Inquiry Center	400 Pennington Avenue P.O. Box 8116 Trenton, NJ 08650-0116		(800) 453 8527 toll free	DuPont Aramids (KEVLAR)	General data on KEVLAR received.
Ciba Composites		5115 East La Palma Avenue, Anaheim, CA 92807, USA		(714) 779 9000 Phone (714) 779 7183 Fax	M10E-AS4D Towpreg	
Hercules Advanced Materials and Systems Co.	Composite Products Group	P.O. Box 98, MS: X11T1 Magna, UT 84044-0098 USA		(801) 251 5372 Phone (800) 443 4237 Phone (801) 251 3268 Fax	Hercules Type AS4 Carbon Fiber tow	
CORDI-GEOPOLYMERE SA		16 Rue Galilee, F-02100 Saint-Quentin, France	Joseph Davidovics	+33 323 626 537 phone +33 323 676 988 fax jd20@calvacom.fr	International coordination and development of geopolimers.	Some basic materials available on Internet. Http://www.insset.u-picardie.fr/geopolymer/pub2.html#8

Craig Ballinger & Associates (coop. with Mitsubishi)		314 AYITO S. Road SE Vienna, VA 22180-2983	Mr. Craig Ballinger	(703) 938 1057 cballinger@aol.com	Cooperates with Mitsubishi. Has a lot of experience (15 years).	Gave contact to Sika (David Wight). Mentioned about tests in ODOT and GA-Tech (dr. Zurick). Sent the materials on Mitsubishi + a few papers.
H.S. Kliger & Associates, Inc.		31 Stratford Circle, Edison , NJ 08820-1815	dr.Howard S. Kliger	(908) 756 0509 phone (908) 754 5292 fax	Commercial applications of Tonen Forca Tow Sheet	He represents Tonen. Talked with secretary. Package sent to S.Y.Park.
Structural Preservation Systems, Inc.		3761 Commerce Drive, Suite 414, Baltimore, MD 21227 OR 2116 Monumental Road Baltimore, MD 21227-1633	Jay Thomas director of sales & marketing	(800) 899 1016 phone (410) 247 1016 phone (410) 247 1136 fax	Commercial applications of Tonen Forca Tow Sheet	The company using TONEN components. Package sent to S.Y.Park
Master Builders Inc.,		23700 Cnagrin Boulevard Cleveland, Ohio 44122-5554		(800) MBT-9990 (216) 831 6910	MBrace Composite Strengthening System based on Tonen Forca Tow Sheet. Commercial applications.	
Hexcel Fyfe Co.		6044 Cornerstone Court West, Suite "C", San Diego, CA 92121-4730	Scott Arnold	(619) 642 0694 (619) 642 0947 E-mail: hexcelfyfe@earthlink.net WWW: http://home.earthlink.net /~hexcelfyfe	Tyfo S Fibrwrap system based on Tonen Forca Tow Sheet. Commercial applications. Numerous strengthenings of bridges and buildings.	Fax sent on March 1st. Replied on March 3rd. Package received on April 4th.
Universities and Research Institutions						
University of Michigan	Civil and Environmental Engineering	2340 G.G. Brown Building 2350 Hayward, Ann Arbor, MI 48109-2125	Prof. Antoine Naaman	(313) 764- 1812 phone (313) 764 4292 fax naaman@engin.umich.edu		Principal Investigator of this research project
Swiss Federal Laboratories for Materials and Testing, EMPA		Uberslandstr. 129 CH-8600, Dubendorf Switzerland	Prof. Urs Meier	01-823 - 55 - 11 phone 01-823 - 62 - 44 fax urs.meier@empa.ch	Free distribution video cassette in PAL and NTSC (English) about post-strengthening of concrete structures with carbon fiber sheets.	Materials & video cassetes received on Feb. 14th.
University of Delaware	Department of Civil Engineering	Newark, DE 19716	Dr Michael Chajes	(302) 831 6056 chajes@ce.udel.edu		Called and E-mailed. Promised to send the materials.
University of Arizona			Hamid Saadatmanesh	(602) 621 2148		Contacted. Package received on Feb. 4th.
University of Wyoming			Prof. Charles Dolan	(307) 766 2857	Starting research. Nothing to send.	Contacted.
University of California		San Diego, California	Prof. Frieder Seible	(619) 534 4640		Called. Adviced to contact Mosen Soltan (Caltrans)
New Jersey Institute of Technology			dr William Spillers	(201) 596 2479	Starting research. Nothing to send.	Contacted.

RUTGERS The State Univ. of New Jersey	Department of Civil Engineering	Rutgers University Box 909 Piscataway, NJ 08855-0909	Prof. P.Balaguru	(908) 445 2232 phone (908) 445 3537 phone (908) 445 0577 fax balaguru@gandalf.rutgers.edu	They cooperate with GEOPOLYMERE	E-mail sent. No reply.
Pennsylvania State University	Architectural Engineering		Prof. Antonio Nanni	(814) 863 2084 phone (814) 863 4789 fax		
West Virginia University	Construction Facilities Center	Morgantown, WV 26506-6101	Prof. Hota V.S. GangaRao	(304) 293 7608		
Georgia Institute of Technology	School of Civil Engineering	Atlanta, GA 30332	Dr. Abdul-Hamid Zureick	(404) 894 2294 phone (404) 894 2278 fax abdul.zureick@gatech.edu		
Lawrence Tech. Institute		Southfield, Michigan	Prof.Nabil F.Grace	(810) 204 2556		
Catholic University	Civil Engineering	Washington D.C.	Dr. Lawrence Bank	(202) 319 4381 phone (202) 319 4499 fax		

UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 3: Behavior of Beams Strengthened For Bending

by

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16. Abstract

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow-sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:
Volume 1 – Summary Report
Volume 2 – Literature Review
Volume 3 – Behavior of Beams Strengthened for Bending
Volume 4 – Behavior of Beams Strengthened for Shear
Volume 5 – Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications.

The part of the investigation dealing with reinforced concrete beams strengthened in bending is described in this volume (volume 3), where the results are also analyzed, compared, and discussed. The experimental program comprised fourteen reinforced concrete T-beams. The test parameters included two levels of steel reinforcement ratio before strengthening, and up to four strengthening levels. Two commercially available strengthening systems were tested, the Sika CFRP plate system (CarboDur), and the Tonen CFRP sheet system. Other selective parameters investigated included two different concrete covers; two conditions of cover preparation, three different end anchorage systems of the glued-on sheets, and pre-loading pre-yielding of the beam prior to strengthening. Conclusions are drawn and some recommendations for design are suggested.

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strengthening systems

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The authors are thankfull to the Sika Corporation (Bob Pisha) for the donation of the Carbodur laminate as well as the Sikadur epoxy material.The partial contribution of Tonen Corporation with the MBrace composite strengthening system, and the help of Howard Kliger are also appreciated.

The opinions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors.

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PREFACE

This project titled: *"Repair and Strengthening of Reinforced Concrete Beams using CFRP Laminates"* is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of the project were:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) glued-on plates for repair and strengthening of concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

The project consisted of 8 tasks as follows:

- A report containing a literature review and a comprehensive synthesis of the latest state of knowledge on the glued -on FRP technique (Task 1);
- Laboratory testing and verification of the selected CFRP glued-on technique according to the proposed experimental program: bending (Task 2), shear (Task 3), freeze-thaw (Task 4), temperature and high cyclic amplitude load (Task 5);
- An interim and final report summarizing the experimental results (Task 6). The interim report will cover the bending and freeze-thaw tests;
- A summary of field specifications and "how to" details for implementation in field applications;
- Guidelines for design based on the experience developed from the experimental work (Task 7);
- Field monitoring of application of the technique to one bridge selected by MDOT (Task 8a);
- Bridge testing before and after application of the glued-on plate (Task 8b to be conducted by professor A. Nowak, U of M)

This report summarizes the experimental program of beams strengthened for bending as per Task 2.

ABSTRACT

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow-sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report is part of a larger investigation on the use of carbon fiber reinforced plastic (CFRP) sheets for repair and strengthening of reinforced and prestressed concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, 2) to synthesize existing knowledge, 3) to identify optimum parameters for successful implementation, 4) to develop procedures for implementation in the field, and 5) to adapt the technique to the specific conditions encountered in the state of Michigan.

The experimental program includes four main parts: 1) tests of RC beams strengthened in bending; 2) tests of RC beams strengthened in shear; 3) freeze-thaw tests of strengthened beams followed by their test in bending; and 4) tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading.

The part of the investigation dealing with reinforced concrete beams strengthened in bending is described in this report, where the results are also analyzed, compared, and discussed. The experimental program comprised fourteen reinforced concrete T-beams. The test parameters included two levels of steel reinforcement ratio before strengthening, and up to four strengthening levels. Two commercially available strengthening systems were tested, the Sika CFRP plate system (CarboDur), and the Tonen CFRP sheet system. Other selective parameters investigated included two different concrete covers; two conditions of cover preparation, three different end anchorage systems of the glued-on sheets, and pre-loading pre-yielding of the beam prior to strengthening. Conclusions are drawn and some recommendations for design are suggested.

Since the plate glued-on technique applies to plain, reinforced and prestressed concrete structures, as well as steel and timber structures, the experience gained during this project and the technology transfer developed should have a much wider impact and should influence a wide range of future applications.

1. GENERAL

Technique of external epoxy-bonded steel plates have been used successfully to increase the strength of girders in existing bridges and buildings. High strength Fiber Reinforced Polymer (FRP) composites are used as an extension of the steel plating method, offering the advantages of composite materials such as non-corrosion, light weight, and unlimited delivery length, thus eliminating the need for joints. FRP sheets or plates may be needed to improve the maximum load capacity and reduce the vertical deflection at service of bridge structures. Also, their use tends to limit the width of cracks and improve their distribution in concrete beams.

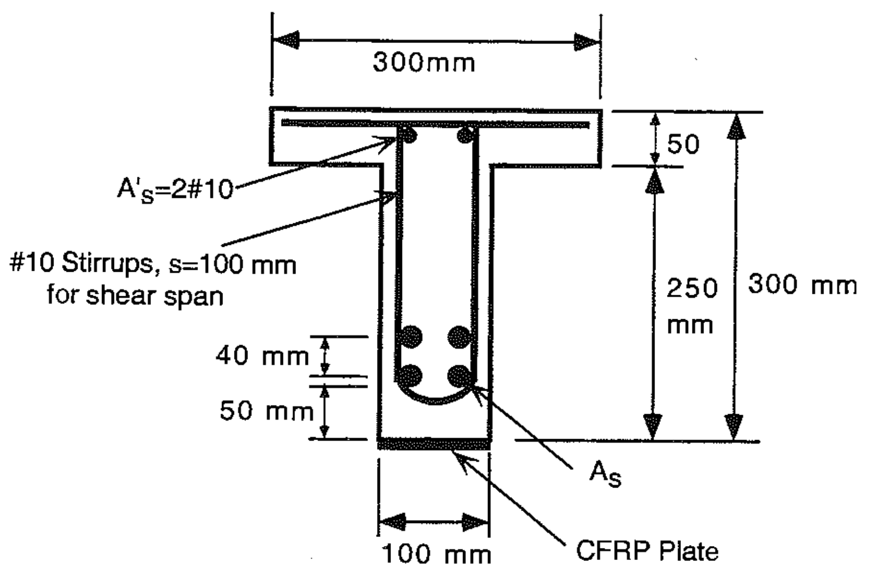
The study described in this report is part of a larger investigation on the use of carbon fiber reinforced plastic (CFRP) sheets for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, 2) to synthesize existing knowledge and develop procedures for implementation in the field, and 3) to adapt the technique to the specific conditions encountered in the state of Michigan.

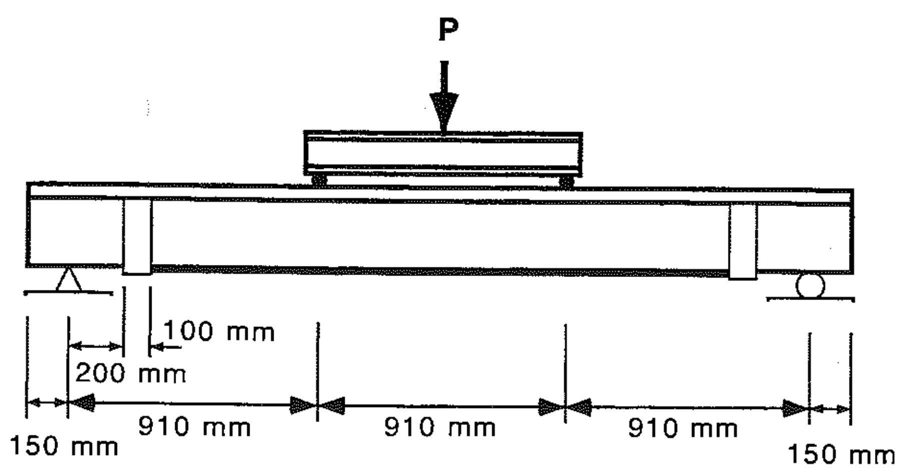
The experimental program includes: 1) tests of reinforced concrete (RC) beam strengthened in bending; 2) tests of RC beams strengthened in shear; 3) freeze-thaw tests of strengthened beams followed by their test in bending; and 4) tests in bending and shear of strengthened beams under low temperature (-29 C) and high amplitude cyclic loading.

The part of the investigation dealing with reinforced concrete beams strengthened in bending is described in this report, where the results are also analyzed, compared, and discussed.

2. EXPERIMENTAL PROGRAM

The experimental program for bending tests comprised fourteen reinforced concrete T beams. The loading arrangement and cross sectional dimensions are shown in Figure 1. All beams were 300 mm deep and 3.0 m long. In most cases, concrete cover for reinforcing steel was kept at 50 mm. To investigate the flexural behavior, a four-point bending test set-up was used. For bending, the CFRP sheet or plate ran along the bottom of a beam extreme tensile fiber as shown in Figure 1. Normally, at its two ends, a 100 mm wide U-shaped anchorage sheet with fibers normal to the axis of the beam was wrapped around in order to provide additional anchorage and minimize the chances of peeling from the ends of CFRP sheets. Before application of the glued-on CFRP sheets or plate, all beams were pre-cracked by pre-loading to about 60% of their ultimate design load, 50 kN and 89 kN for the beams with the steel reinforcement ratios $0.27\rho_{max}$ and $0.54\rho_{max}$, respectively. This is to simulate actual condition of cracked reinforced concrete beams at the time the strengthening system is applied.





Throughout the experimental study, particular effort was placed at observing the type of failure and understanding its mechanisms. Also, the applied load, corresponding deflection, as well as the strains of reinforcing bars and CFRP sheet or plate were measured.

2.1 Test Parameters

A number of test parameters were initially proposed by the research team and further refined after discussion with the Technical Advisory Group of the project. These parameters and related experimental variables for the bending tests are summarized in Table 1 and illustrated in Figure 2.

The decision was made to consider two different reinforcement ratios corresponding approximately to one third and two thirds the maximum reinforcement ratio (ρ_{max}) recommended by the AASHTO Code, sixteenth edition. Computations of reinforcement ratios and selection of reinforcing bars were carried out on the basis that the concrete compressive strength, obtained from cylinder tests, would be 35 MPa. Other parameters were then selected as shown in Table 1a. However, the beams were prepared in factory in a precast concrete plant and were steam cured; although the specified compressive strength of concrete was 35 MPa, the actual compressive strength at time of testing averaged about 56 MPa. Thus back-calculation was carried out to lead to the actual test parameters described in Tables 1a and 1b (see Appendix A for detailed calculations). These parameters should be considered the real parameters of the test program. In addition, the actual value for yield strength of the reinforcing bars was also used for this back calculation.

The test parameters included the existing steel reinforcement ratio before strengthening, and the strengthening level. For each steel reinforcement ratio, a control beam was tested and compared with CFRP glued-on beams having different strengthening levels. Beams No. 1 and No. 5 were control beams with steel reinforcement ratios of $0.27\rho_{max}$ and $0.54\rho_{max}$. The maximum steel reinforcement ratio, ρ_{max}, was defined as per the AASHTO Code to represent 75% of the balanced ratio. The strengthening level (i.e., the number of CFRP sheets, if more than one) was determined assuming that (except for beam 8-1) the total reinforcement of steel and CFRP will lead to a moment resistance not exceeding the moment corresponding to the maximum reinforcement ratio allowed for the beam by AASHTO or ACI, assuming beams with reinforcing bars only.

Two strengthening systems were tested: 1) the Sika CFRP plate system (CarboDur), and 2) the Tonen CFRP sheet system. The characteristics of these two systems are described in Section 2.2.2. The Tonen system was used throughout except for two beams, Beams No. 8 and 8-1. For Beam No. 8, the glued-on Sika CFRP plate had a width of 40 mm, which is equivalent in tensile strength to 2 layers of Tonen CFRP sheets (Forca Tow sheet). Following testing, Beam No. 8 was in very good shape even after the interfacial shear failure of concrete. There was no spalling of concrete cover even though the reinforcing bars had yielded and the 40 mm wide CFRP plate was completely delaminated. Beam No. 8 was later re-used as Beam No. 8-1, this time with a CFRP plate 100 mm wide to study different bond widths and strengthening levels.

For one selected set of parameters, two different concrete covers and cover conditions were investigated to study the influence of concrete cover on strengthening effect and mode of failure. Beam No. 12 had only a 25 mm concrete cover compared to the normal clear cover of 50 mm of the other beams. Beam No. 9 also had an initial 25 mm clear concrete cover to which additional 25 mm repair mortar cover (Sika Top 122 Plus) was added to simulate damaged concrete in real beams.

To evaluate different anchorage systems, three different anchorage conditions were provided. Beam No. 10 had extended end anchorage which means that the glued-on CFRP sheets were extended up to about 50 mm from the supports, without adding the U-shaped wrapped-around end anchorage.

Beam No. 14 had neither a wrapped end anchorage nor an extended end anchorage. All other beams strengthened with Tonen sheets had, at both ends, a 100 mm wide U-shaped wrapped-around end anchorage perpendicular to the longitudinal CFRP sheets. The beam with strengthened with the Sika system (8,8-1) did not have a wrapped end anchorage.

Beam No. 11 was pre-loaded beyond yielding of steel reinforcing bars, to about 180 kN, to investigate the influence of loading history before the application of CFRP plate. The permanent deflection and maximum crack width at unloading in the pre-cracked beam were 25 mm and 0.9 mm, respectively.

For all beams except for Beam No. 13, the concrete surface to be glued on was prepared, for better bond, by grinding with a disk grinder according to the recommendations of the supplier of the strengthening system used. For Beam No. 13, the surface of concrete was simply cleaned with a vacuum cleaner and wiped with a clean cloth to remove any dust.

2.2 Preparation of Test Beams

2.2.1 Concrete

The test beams were supplied by a precast concrete manufacturer, according to the design specifications. Portland cement Type-III cement, natural sand, and crushed limestone aggregates with a maximum size of 12.5 mm were used for the concrete. The mix proportions of the concrete as provided by the supplier are presented in Table 2.

Table 1a Parameters and variables for the bending tests($f'c$=55.2 MPa).

Beam No.	Test parameter	Reinforcement ratio, ρ	$A_{s(used)}$ mm^2	M_n/M_{max} %	Force Tow sheet FTS-C1-30	Strengthening ratio[1] % ()[2]	CarboDur strip (1 layer)	Strengthening ratio[1] % ()[2]
1	Steel reinforcement ratio & Strengthening level	$0.27\rho_{max}$	2#10 2#13 A_s=400	29	0	0 (29)		
2					1 layer	12 (41)		
3					2 layers	24 (52)		
4					4 layers	47 (75)		
5		$0.54\rho_{max}$	4#16 A_s=800	57	0	0 (57)		
6					1 layer	12 (68)		
7					2 layers	24 (80)		
8	Different system (Sika)	$0.54\rho_{max}$	4#16 A_s=800	57			width= 40 mm	22 (78)
8-1		$0.54\rho_{max}$	4#16 A_s=800	57			width= 100 mm	60 (113)
9	Repaired concrete _over	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
10	Extended end _nchorage	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
11	Pre-loading	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
12	Concrete cover depth 25 mm	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	22 (78)		
13	Cleaned surface	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		
14	No anchorage	$0.54\rho_{max}$	4#16 A_s=800	57	2 layers	24 (80)		

Note: 1: $M_{FRP}/M_{max.}$x100

2: $(M_{As}+M_{FRP})/M_{max.}$x100, $M_{max.}=M_n$ (when $As=As_{max}$), (f'_c=55.2 MPa, f_y=455 MPa, A_{smax}=1490 mm^2)

All the above values are calculated

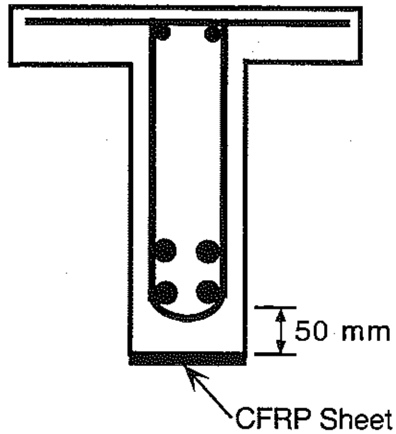
Table 1b Test parameters and variables for bending test ($f'c$= 55.2 MPa)

Beam No.	Test parameter	Reinforcement ratio, ρ	$A_{s(used)}$ mm^2	Effective depth (mm)	Forca Tow sheet FTS-C1-30	Strengthening ratio[1] % ()[2]	CarboDur strip (1 layer)	Strengthening ratio[1] % ()[2]
1	Steel reinforcement ratio & strengthening level	$0.27\rho_{max}$	2#10 2#13 A_s=400	d_s=235 d_f=305	0	0 (29)		
2					1 layer	41 (41)		
3					2 layers	81 (52)		
4					4 layers	160 (75)		
5		$0.54\rho_{max}$	4#16 A_s=800	d_s=227 d_f=305	0	0 (57)		
6					1 layer	21 (68)		
7					2 layers	41 (80)		
8	Different system (Sika)	$0.54\rho_{max}$	4#16 A_s=800	d_s=227 d_f=305			width= 40 mm	39 (78)
8-1		$0.54\rho_{max}$	4#16 A_s=800				width= 100 mm	100 (113)
9	Repaired concrete _over	$0.54\rho_{max}$	4#16 A_s=800	d_s=227 d_f=305	2 layers	41 (80)		
10	Extended end _nchorage	$0.54\rho_{max}$	4#16 A_s=800		2 layers	41 (80)		
11	Pre-loading	$0.54\rho_{max}$	4#16 A_s=800		2 layers	41 (80)		
12	Concrete cover depth 25 mm	$0.54\rho_{max}$	4#16 A_s=800	d_s=227 d_f=279	2 layers	37 (78)		
13	Cleaned surface	$0.54\rho_{max}$	4#16 A_s=800	d_s=227 d_f=305	2 layers	41 (80)		
14	No anchorage	$0.54\rho_{max}$	4#16 A_s=800		2 layers	41 (80)		

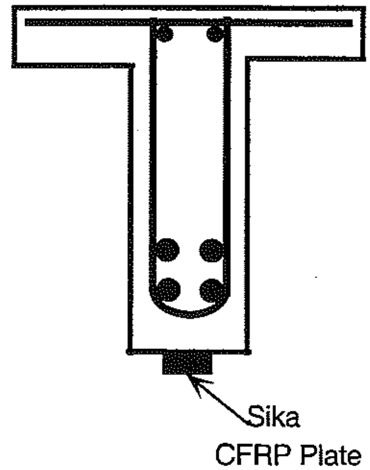
Note: 1: $(M_{(As+FRP)}-M_{As.}(control))/M_{As.}(control)$ x100, based on the assumption of CFRP tensile failure

2: $(M_{As}+M_{FRP})/M_{max.}$x100, $M_{max.}=M_n$(when $\rho=\rho_{max.}$), (f'_c=55.2 MPa, f_y=455 MPa, A_{smax}=1490 mm^2)

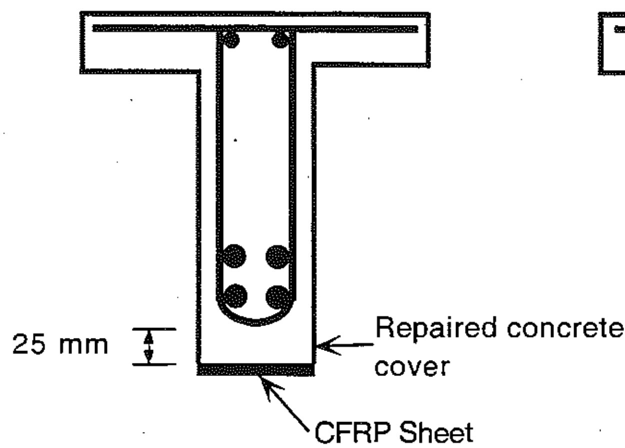
All the above values are calculated



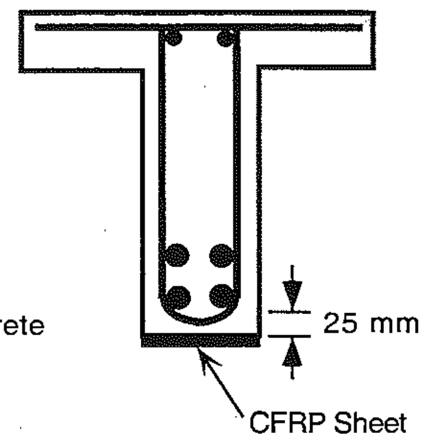
<u>Normal concrete cover</u>



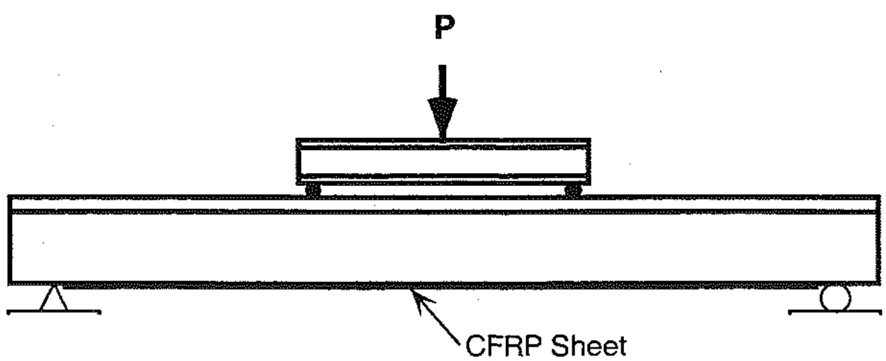
<u>Different system</u>



<u>Repaired concrete cover</u>



<u>Concrete cover depth</u>



<u>Extended end anchorage</u>

Table 2 Mix proportions of concrete.

Materials	Weight Ratio to Cement	Batch weight Kg-f/m^3
Type-III cement	1	418
Water	0.38	160
Sand (2S)	1.50	626
Coarse aggregate (26-A)	2.54	1062
Plasticizer (400N)	0.13	54
Air entrainer (AE-90)	0.035	15

The properties of the fresh concrete mixture were as follows;

- Air content : 6 %
- Slump: 100 mm
- Concrete unit weight: 22.3 kN/m^3

Table 3 Compressive strength of concrete and age of test beams.

Batch No.	Age of concrete, day	Compressive strength of concrete, MPa	Test beam No.	Age of test beam, days
1	0.75*	32.1	8, (8-1)	86, (129)
	85	56.7	13	84
2	0.75*	30.8	6	29
	29	54.4	14	80
3	0.75*	28.3	5	77
	83	54.0	10	78
4	0.75*	28.0	7	28
	29	55.1	11	141
5	3	43.9	3	74
	81	58.8	4	73
6	0.75*	25.8	1	64
	78	51.9	2	65
7	0.75*	34.3	9	125
	77	55.5	12	77

* Steam cured concrete cylinders tested at 18 hours.

Note: Number of cylinders = 7 batches x 4 ea. = 28; 2 beams/batch = 14 beams.

Since the beams were steam cured, their average compressive strength obtained from two cylinder tests was measured at 18 hours (except for beam 5) and is given in Table 3. Also given in Table 3, is the average two-cylinders compressive strength of the concrete matrix at about the same time of testing for each beam. Difference in time is considered not significant. On the average, the compressive strength was about 55.2 Mpa.

2.2.2 CFRP Sheets

For strengthening of the test beams, the Forca Tow Sheet (Tonen CFRP sheet) supplied by Master Builders as part of the MBrace system (MB CF 130) and the CarboDur plate (Sika CFRP plate) with the corresponding epoxy adhesive (MBrace Saturant for the MBrace system and Sikadur 30 for the Sika system) of the same system were used. The properties of the materials in the two systems are summarized in Table 4. Values were provided by the manufacturers. According to the manufacturer, the tensile stress-strain curve of the CFRP sheet or plate is linear elastic up to failure.

2.2.3 Reinforcing Bars

The steel reinforcing bars used had a diameter of No. 10, No. 13, and No. 16 and were of Grade 400 corresponding to a minimum yield strength of 410 MPa with a tensile modulus of 200 GPa. Table 5 presents the actual yield strengths and elastic moduli obtained from direct tension tests carried out in this study on samples of bars.

2.2.4 Fabrication of Test Beams

All beams were fabricated in a precast concrete plant and delivered to the test laboratory with a number of additional cylinders for testing the concrete compressive strength. All test beams had four longitudinal reinforcing bars, placed in two rows, two in the lower row at a center distance of 60 mm from the bottom fiber of concrete, and two in the upper row with a center to center distance of 38 mm from the lower row. For each beam, two strain gages were attached on the lower two reinforcing bars by the research team before assembling the reinforcement cage. All beams had 50 mm deep clear concrete cover from the bottom fiber except for Beam No. 12, which had 25 mm deep clear concrete cover.

Table 5 Yield strength and elastic modulus of reinforcing bars as tested.

Reinforcing bar		Yield stress MPa	Elastic modulus GPa
No. 10	#10-1	516	196
	#10-2	492	201
	#10-3	507	201
	Average	505	199
No. 13	#13-1	427	172
	#13-2	431	187
	#13-3	424	–
	Average	427	175
No. 16	#16-1	452	190
	#16-2	450	201
	#16-3	454	184
	Average	452	192

Table 4 Properties of CFRP sheets (plates) and adhesives.

Supplier		Tonen			Sika
System		Forca Tow Sheet			CarboDur
CFRP Sheet	Type	FTS-C1-20	FTS-C1-30 (MBrace CF 130)	FTS-C5-30	CarboDur Strip
	Tensile Strength, N/mm GPa	383 3.48	574 3.48	487 2.94	2868 2.40
	Tensile Modulus, kN/mm GPa	25.4 228	38.5 228	61.3 372	178.6 150
	Thickness[1], mm	0.11	0.17	0.17	1.19
	Elongation, %	1.5	1.5	0.8	1.4
	Width, mm	500			50, 80, 100
	Length, m	Unlimited			Up to 250
Epoxy	Type	FR-E3P	MBrace saturant	FR-E3PW	Sikadur 30
	Application temperature, °C	Standard	10-38	Winter	≥4
	Tensile strength, MPa		78		24.8
	Elastic Modulus, GPa				4.48
	Elongation, %				1
	Shear strength, MPa				24.8
	Shrinkage				0.0004
	Pot Life, min.	40	110	20	70
	Viscosity, cps	20,000	20,000	10,000	
Primer	Type	Standard, Summer, Winter, Penetrative, Summer damp surface, Winter damp surface			No Primer

1: Total cross sectional area of fibers per inch.

To ensure flexural failure, sufficient stirrups designed according to AASHTO code were provided for all beams. Two-leg stirrups (closed with extended ends) made of 10 mm reinforcing bar were placed at a spacing of 100 mm and 200 mm within the shear span and constant moment span of the test beam, respectively. Figure 3 shows the casting of concrete of the test beams.

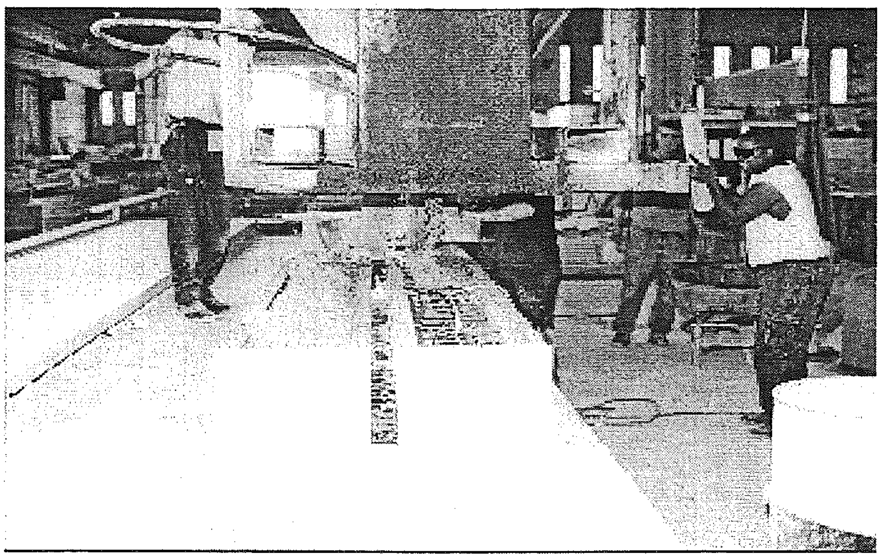


Figure 3 Casting of concrete of test beams.

2.2.5 Preparation of Concrete Surface for Bond

For the preparation of concrete surface to be glued on, two methods, steel brushing and disk grinding, were initially proposed to the supplier of the strengthening system. Of the two methods, the disk grinding method was recommended by the system supplier, after visual check of the surface substrate. The concrete surface was ground enough to remove laitance and show the open texture of the aggregates. After grinding, the dust was removed by brushing and vacuum cleaning the surface. Generally, the surface to be coated with the epoxy resin was even (level) without any roughness or formwork marks larger than 1.0 mm. To evaluate a different surface condition, the bottom surface of Beam No. 13 was simply wiped with a clean cloth and vacuumed without any disk grinding.

2.2.6 Gluing CFRP Sheet or Plate

The Forca Tow (Tonen) sheets and CarboDur (Sika) plates were cut to proper lengths using a sharp blade and a disk cutter, respectively. The CFRP sheet or plate was wiped with a clean cloth before starting application, in order to remove soiling as well as carbon dust.

The adhesives components were mixed according to the technical data sheet provided by the system supplier. Tonen epoxy resin was so sticky that mixing by hand was not easy. After first opening of the bucket, Tonen epoxy resin was so hardened from the surface that uniform mixing was very difficult because of hardened lumps and high viscosity. Sika epoxy was mixed in uniform consistency without any difficulties. It was like cement mortar with very low viscosity.

For the Tonen strengthening system, the primer and epoxy adhesive were applied using a roller similarly to the commercial specifications. The application of the adhesive using a roller, to form a layer of uniform thickness, was not easy because of its high viscosity. To press the CFRP sheet into the adhesive, a non-stick paper was placed between the CFRP sheet and the roller, and a harder roller was utilized. Figure 4 shows the gluing of the Tonen CFRP sheet on the soffit of a beam.

For the Sika strengthening system, a trowel was used to apply the epoxy adhesive to the surface of beam soffit and CFRP plate. To press CFRP plate into the epoxy adhesive, a hard roller was used forcing out the adhesive from the plate sides.

Figure 4 Gluing of CFRP sheet on the soffit of a beam.

2.3 Data Acquisition and Test Procedure

Figure 5 shows the instrumentation layout for the bending tests. A computer based data acquisition system (Megadack System) was used to measure load and deflection as well as strains of the steel reinforcing bars and the CFRP sheet or plate.

Each test beam was loaded monotonically up to failure using displacement control at a loading rate of 0.13 mm per second by the Instron loading machine having a capacity of 450 kN. Each beam was pre-loaded to about 8.9 kN before testing to remove any residual stress and deformation in the test beam and stabilize the instrumentation. At every 8.9 kN interval, loading was temporarily stopped to mark cracks. The following data was obtained every second by the data acquisition system: (1) load and deflection from the Instron loading machine, (2) strains of the reinforcing bars at midspan, (3) strains of the CFRP sheet or plate at the middle and both ends of span, and (4) deflections from the LVDTs at mid span.

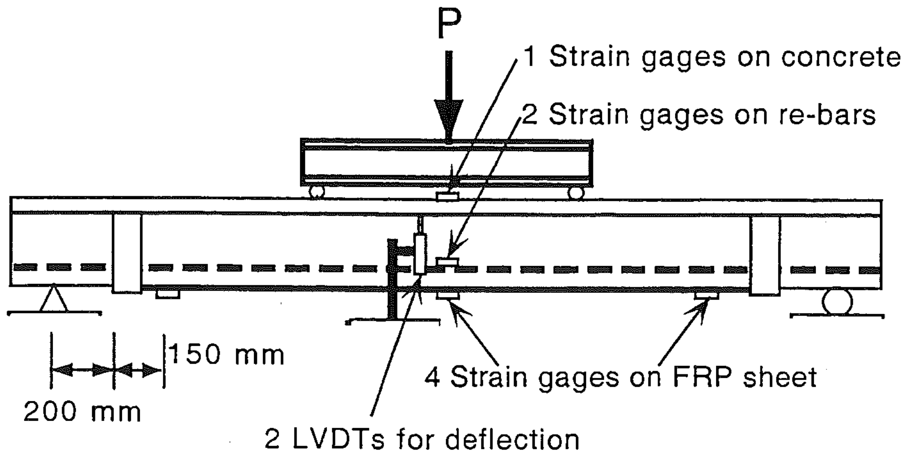

P
1 Strain gages on concrete
2 Strain gages on re-bars
150 mm
200 mm
4 Strain gages on FRP sheet
2 LVDTs for deflection

Table 6 Summary of test results of bending test

Beam No.	Test parameter	Reinforcement ratio	No. of CFRP layer	Failure mode[1]	Ultimate load kN	Strengthening ratio[2] % ()[3]	Ultimate deflection mm
1	Steel reinforcement ratio & strengthening level	$0.27\rho_{max}$	0	Steel yielding	114.8	0	164
2			1	CFRP rupture	135.0	18 (41)	83
3			2	Interface failure	140.4	22 (81)	56
4			4	Interface failure	160.7	40 (160)	49
5		$0.54\rho_{max}$	0	Steel yielding	188.0	0	88
6			1	Interface failure	209.9	12 (21)	77
7			2	Interface failure	222.0	16 (41)	51
8	Different system (Sika)	$0.54\rho_{max}$	40 mm	Interface failure	209.2	11 (39)	46
8-1			100 mm	Interface failure	250.9	33 (100)	41
9	Repaired concrete cover		2	Interface bond failure	208.2	11 (41)	73
10	Extended end anchorage			Interface failure	220.4	17 (41)	59
11	Pre-loading			Inter-laminar failure	226.2	20 (41)	119
12	Concrete cover depth			Interface failure	221.2	18 (37)	69
13	Cleaned surface			Interface failure	230.8	23 (41)	60
14	No anchorage			Interface failure	215.1	14 (41)	57

1: Steel yielding: Compression failure of top concrete long after reinforcement yielding
CFRP rupture: Tensile failure of CFRP sheet
Interface failure: Interfacial shear failure of concrete just above the epoxy adhesive.
Interface bond failure: Interfacial bond failure between the repair mortar and the existing concrete
Inter-laminar failure: Inter-laminar shear failure between glued-on CFRP sheets

2: Actual strengthening ratio compared to control beam (Beam No. 1 or No. 5)

3: Design strengthening ratio compared to control beam based on the assumption of CFRP tensile failure (see Table 1b)

In their failure modes, Beam No. 2 failed by the tensile failure of CFRP sheet and Beams No. 3, and No. 4 failed by interfacial shear failure of the concrete (delamination) just above the epoxy adhesive. On the other hand, the control beam, Beam No. 1, failed by compression failure of the concrete in the top flange long after yielding of the steel reinforcing bars, which is a typical failure mode in conventional under-reinforced concrete beams. The CFRP sheets in the strengthened beams inhibited the growth of large cracks, which had occurred in the control beam, by leading to a better crack distribution with smaller crack widths and spacing. This can help to protect reinforcement from further corrosion.

In Beam No. 2, which failed by the tensile failure of CFRP sheet, the glued CFRP sheet was ruptured piece by piece (strip by strip) continuously over the length. Figure 6 shows the tensile failure of CFRP sheet observed in Beam No.2. In this failure mode, unlike the beams that failed by interfacial shear failure of concrete, there were no pieces of concrete cover spalled off from the beam. In other cases, interfacial shear failure of concrete is more likely to occur between the surface of concrete and the CFRP laminates.

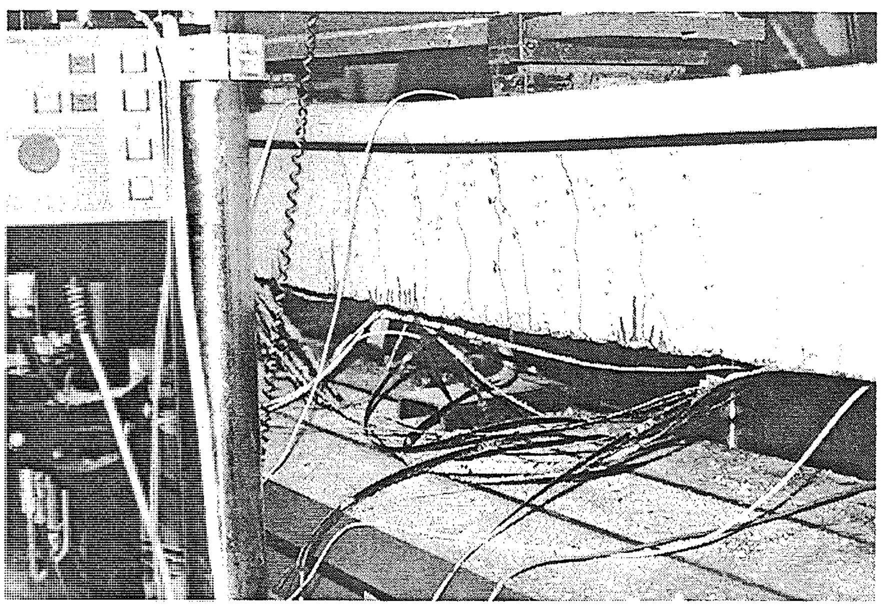


Figure 6 Tensile failure of CFRP sheet in Beam No. 2

In Beams No. 3 and No. 4 which failed by interfacial shear failure, the glued-on CFRP sheet was delaminated along the interface between the surface of concrete and CFRP sheet as shown in Figure 7. The epoxy adhesive on the CFRP sheet tore out the concrete just above the interface. The delamination seems to have started at a crack below the loading point where bending moment was maximum and suddenly propagated to the end of the CFRP sheet. Figures 8 and 9 prove that the delamination did not start from the end of the CFRP sheet, because the strains of the CFRP sheet at their ends only slightly decreased when the applied load suddenly dropped from its maximum value at onset of delamination. On the other hand, the strains in the middle of the CFRP sheet significantly decreased at onset delamination. Additional supporting evidence for this argument is that the U-shaped wrapped end anchorage in Beam No. 7 did not significantly improve the ultimate load capacity in comparison to that of Beam No. 14, which had no anchorage. When the delamination occurred on one side of the beam, the impact energy released from the tensioned CFRP sheet tore out the concrete cover already cracked vertically by flexure along the longitudinal reinforcing bars in the constant bending moment zone (Figure 7).

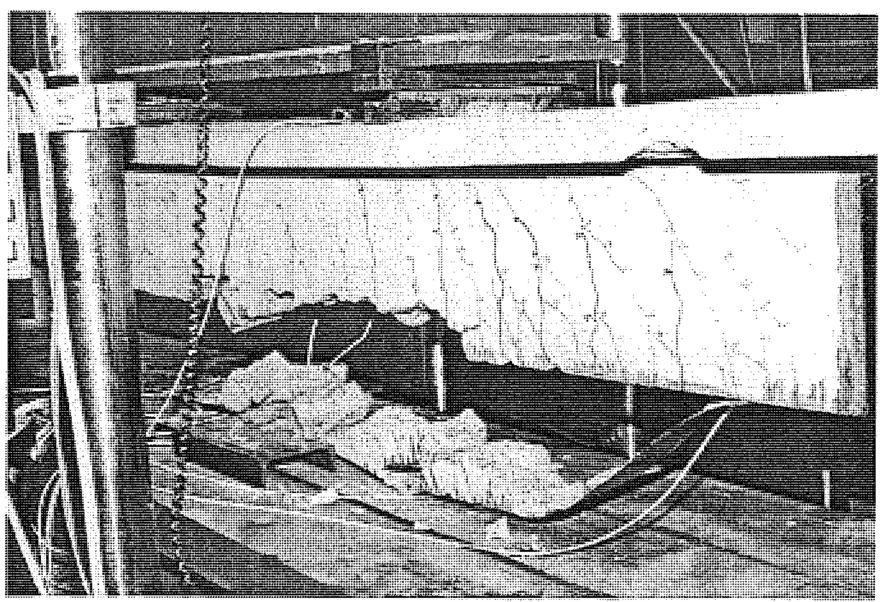


Figure 7 Interfacial shear failure of concrete in Beam No. 3.

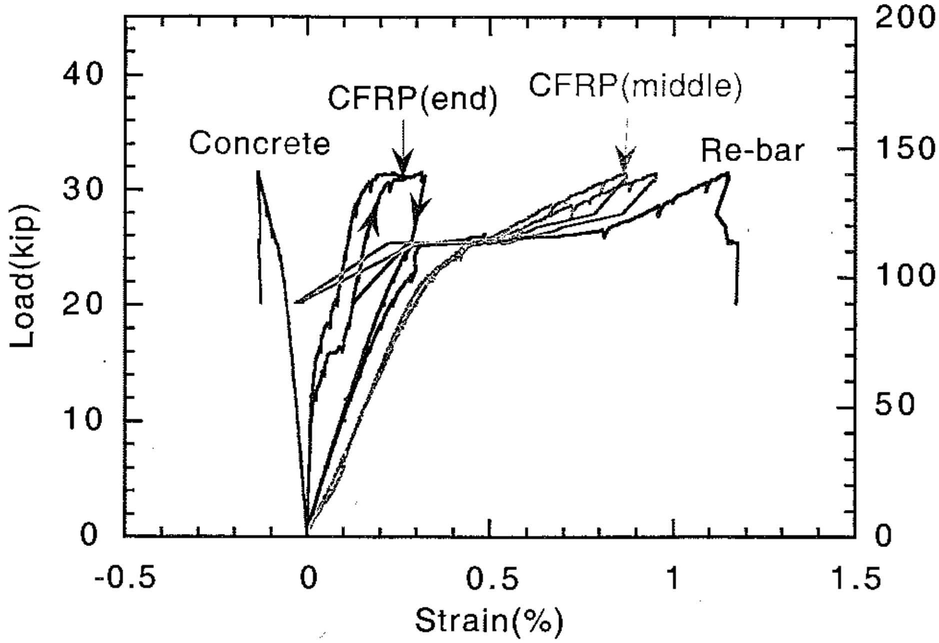


Figure 8 Load-strain curves of Beam No. 3.

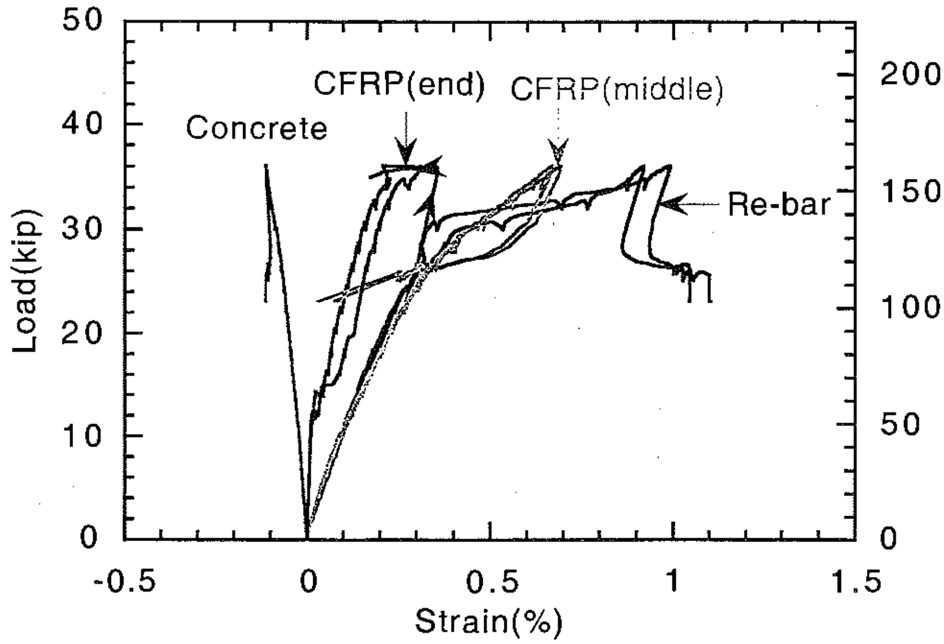


Figure 9 Load-strain curves of Beam No. 4.

Figure 10 compares the load-deflection curves of the control beam and the strengthened beams with different strengthening level (i.e., one, two, and three layers of CFRP sheets). As shown in the figure, the ultimate load considerably increased with an increase in strengthening level, while the ultimate deflection significantly decreased. Discussion about these increases in strength and decreases in deflection is covered in Section 3.1.3 Strengthening Level with the series of beams having a maximum reinforcement ratio of $0.54\rho_{max}$. The strengthened beams were stiffer than the control beam before and after reinforcement yielding. The control beam, Beam No. 1, was very ductile as expected in reinforced concrete beams with low reinforcement ratio.

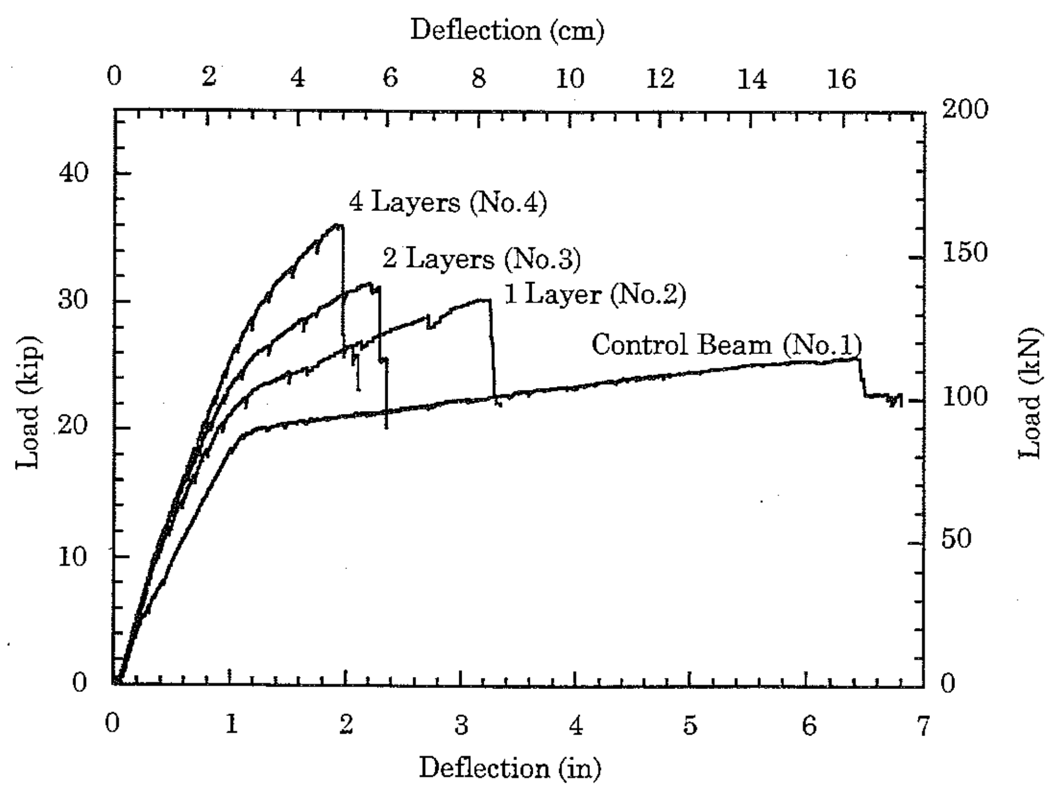


Figure 10 Load-deflection curves of beams with reinforcement ratio, $0.27\rho_{max}$, for different strengthening levels.

Figures 11 and 12 show respectively the curves for beam load and beam deflection versus the strain in the tensile reinforcing bar, at different strengthening levels. All load-strain responses of reinforcing bars were roughly bi-linear with a yield strain of about 0.3%, which was slightly larger than the 0.24% expected yield strain. In the elastic range before yielding, the reinforcing bars in the beams strengthened using CFRP sheets had less strain. In other words, the reinforcing bars were less stressed at a given deflection. From Figure 12, it can be observed that the deflection of the control beam linearly increases as the strain of the reinforcing bar increases. However, the deflection of the beams strengthened with CFRP sheets increased linearly only prior to reinforcement yielding.

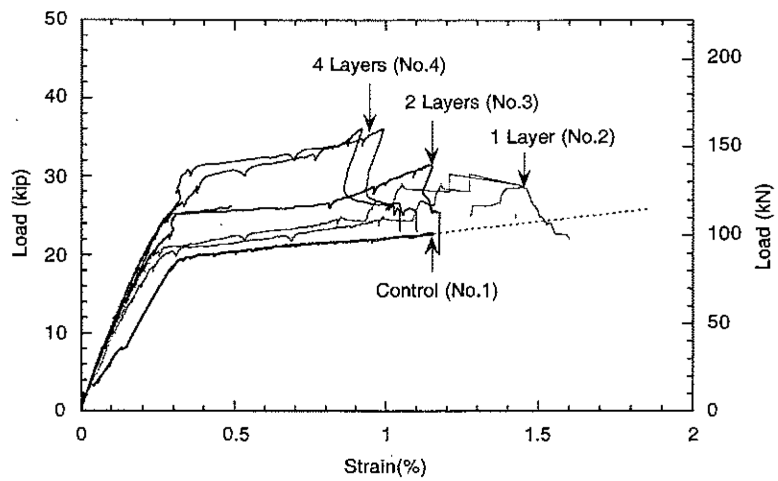


Figure 11 Load-strain curves of reinforcing bar.

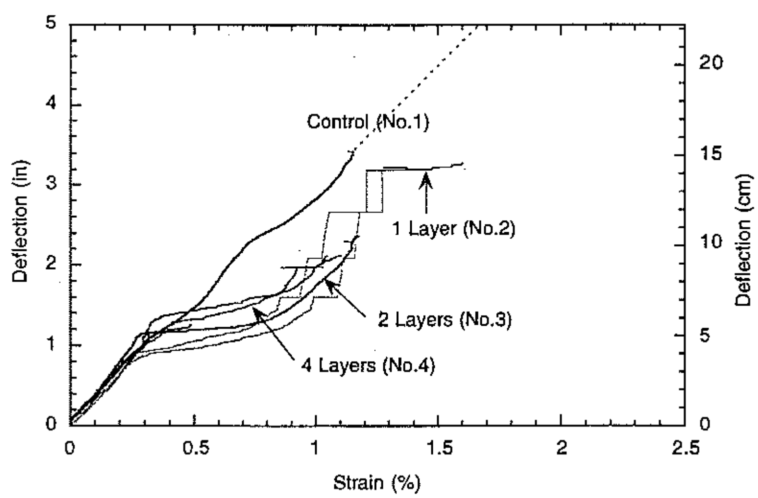


Figure 12 Deflection-strain curves of reinforcing bar.

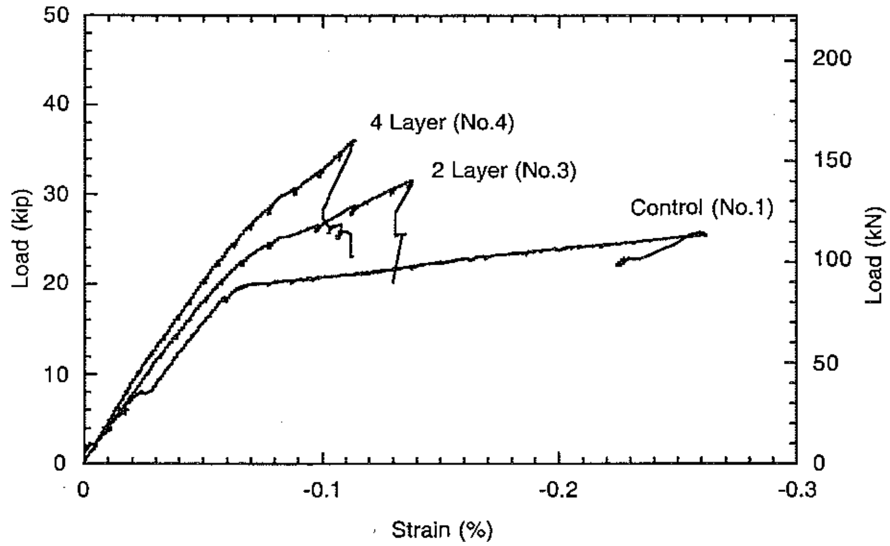


Figure 13 Load-strain curves of concrete in top flange.

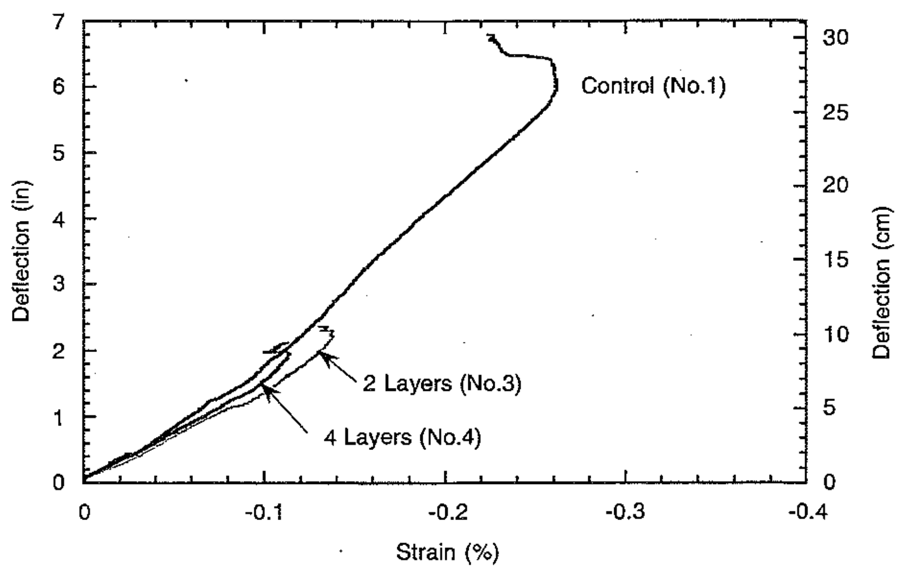


Figure 14 Deflection-strain curves of concrete in top flange.

Figures 13 and 14 show the load-strain and deflection-strain curves of the concrete top flange. Load-strain and deflection- strain curves were roughly bi-linear and linear, respectively. In the control beam, the concrete in the top flange failed by compression at a compression strain of about 0.26%. In Beam No. 3 and No. 4, the compression strains of concrete in the top flange were about 0.11 % and 0.14 % at failure by delamination of CFRP sheet, respectively. As can be seen in

Figure 13, the slope of load-strain curves increased with an increase of strengthening level. However, the deflection-strain relationship showed little difference in their slopes.

Figures 15 and 16 show the load-strain and deflection-strain curves of the CFRP glued-on sheets. As shown in Figure 15, the strain (or stress) in the CFRP sheet in Beam No. 3 and 4 was lost very rapidly following delamination of the sheet. In Beam No. 2, the CFRP sheet ruptured at about 1.45% strain, which was close to the specified failure strain of 1.5%. The deflection-strain relationship was linear up to failure with little difference in slope for different strengthening ratios. This observation implies that the strengthening effect is proportional to the strengthening level (i.e., the number of CFRP sheets). This interpretation is also confirmed by the ultimate load-strengthening level curves described in Section 3.1.3 :Strengthening Level.

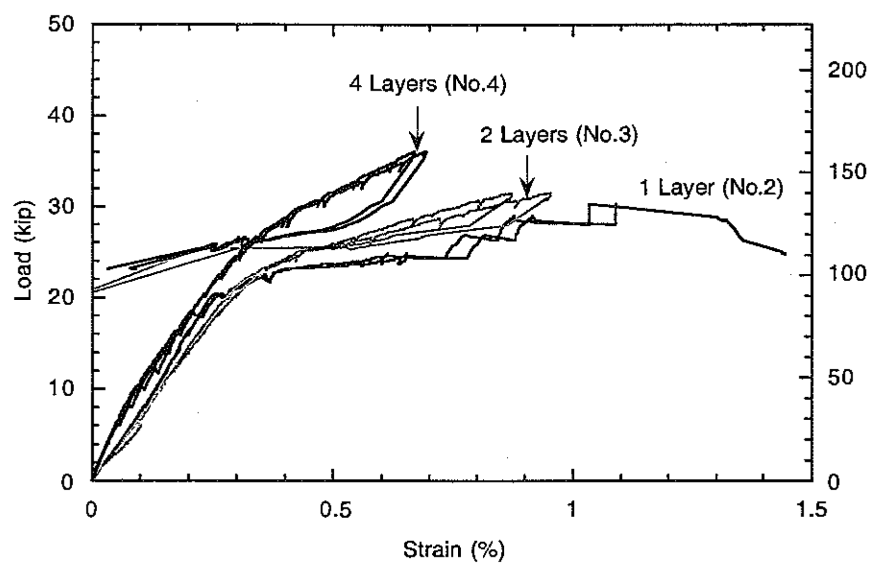


Figure 15 Load-strain curves of CFRP sheet.

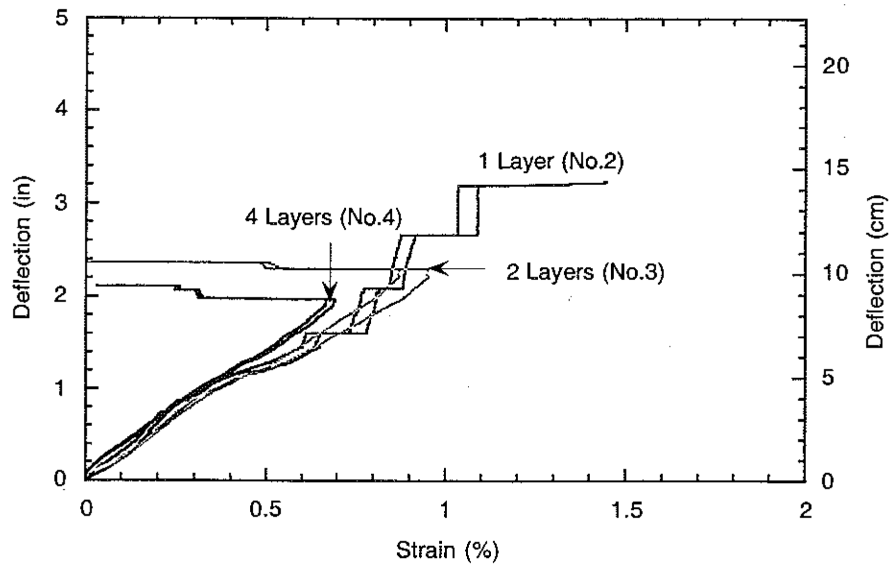


Figure 16 Deflection-strain curves of CFRP sheet.

3.1.2 Reinforcement Ratio $0.54\rho_{max}$

To study the influence of existing steel reinforcement ratio and strengthening level, the second series of beams with a maximum steel reinforcement ratio, $0.54\rho_{max}$, was tested. Beams No. 6 and No. 7 were strengthened using CFRP sheets , having a tensile strength of 3,480 MPa and a tensile modulus of 228 GPa. One layer of the CFRP sheet provided maximum tensile load of 574 N/mm width.

Upon testing, Beams No. 6 and 7 failed by interfacial shear failure of concrete (delamination) similarly to Beams No. 3 and No. 4. of the first series of tests. Control beam, Beam No. 5, failed by compression failure of the concrete in top flange after yielding of the reinforcement, as expected.

In Beam. No. 6 strengthened with one layer of CFRP, only one small strip of the CFRP sheet (about one fifth of it) ruptured at the ultimate state and, shortly thereafter, the remaining part of the CFRP sheet delaminated with only two pieces of concrete spalled off from the concrete cover as shown in Figure 17. This combination of tensile rupture and delamination of CFRP sheet indicates that the CFRP sheet almost reached its failure stress (or strain) in tension, implying that it was fully effective in terms of strengthening. This conclusion was confirmed later from analysis of the load-strain curves of CFRP sheet. The measured strain at the ultimate load was about 1.4%, which is slightly less than the specified strain of 1.5% (Figure 21).

Figure 18 compares the load-deflection response curves of the control beam and the beams strengthened with one and two layers of CFRP sheets. Figures 19 to 22 show the corresponding load-strain and deflection-strain curves of the reinforcing bar and the CFRP sheets. The analysis of these results is very similar to that carried out for the beams with a maximum steel reinforcement ratio, $0.27\rho_{max}$, even though their numerical values are different.

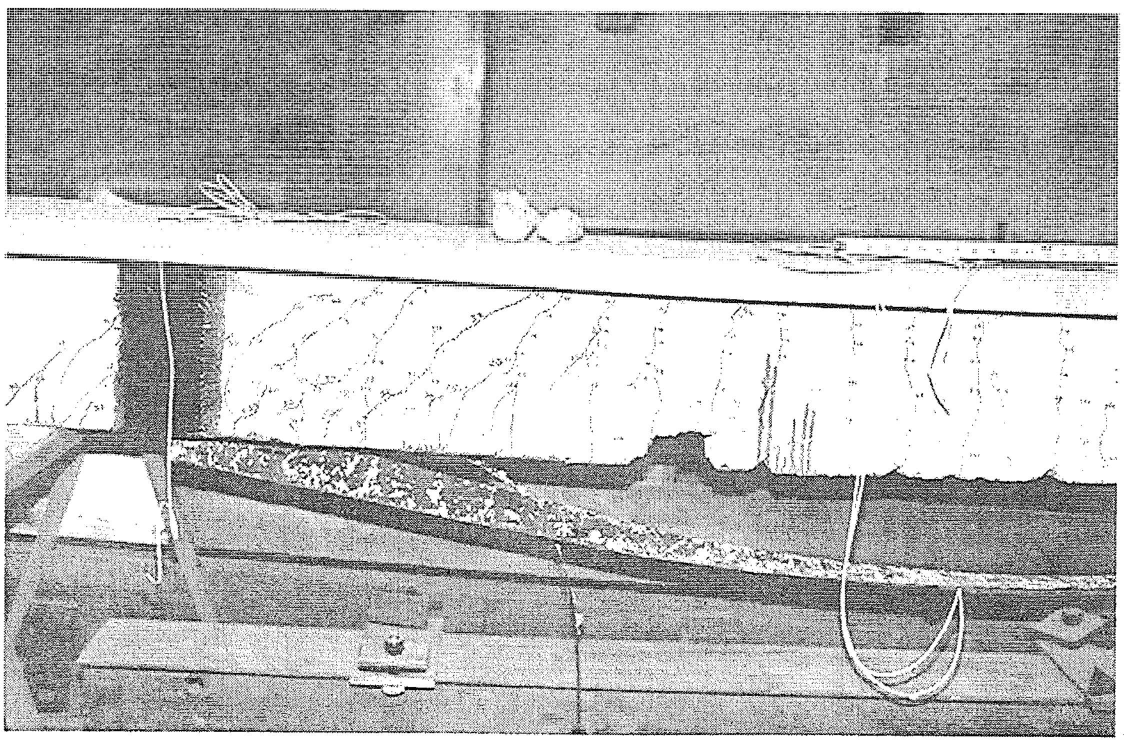


Figure 17 Tensile rupture and delamination of CFRP sheet in Beam No. 6.

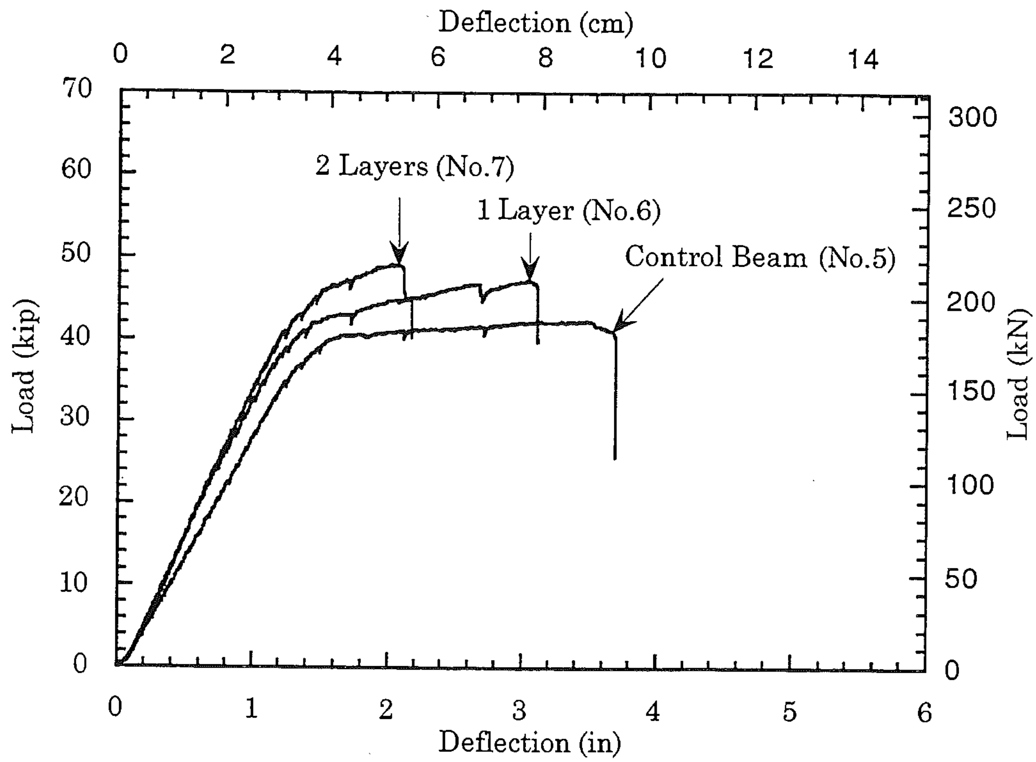


Figure 18 Load-deflection curves of beams with reinforcement ratio, $0.54\rho_{max}$, at two strengthening levels.

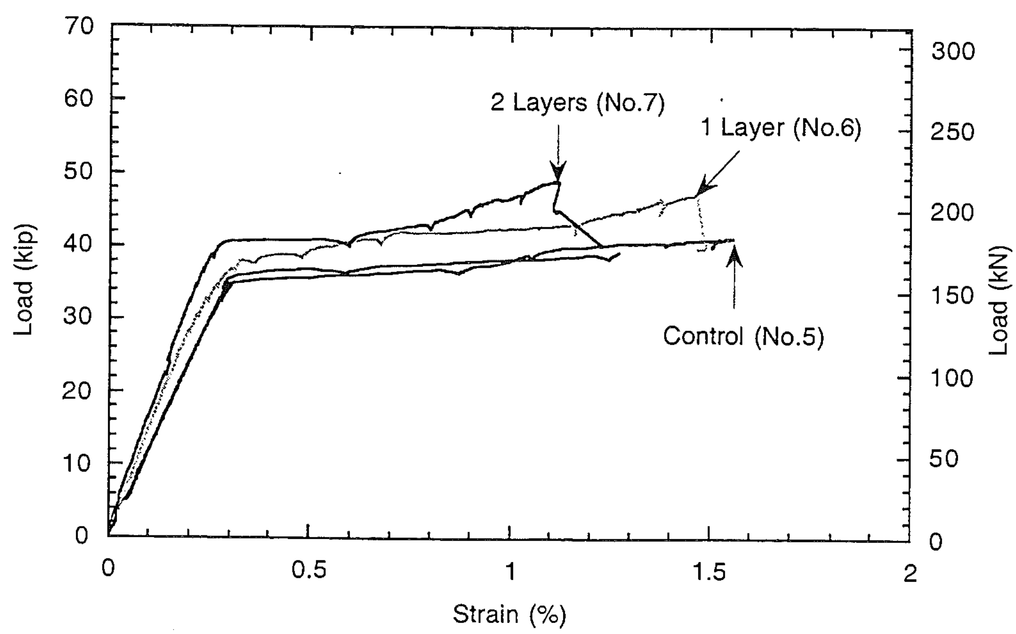


Figure 19 Load-strain of reinforcing bar of beams with reinforcement ratio, $0.54\rho_{max}$, at two strengthening levels.

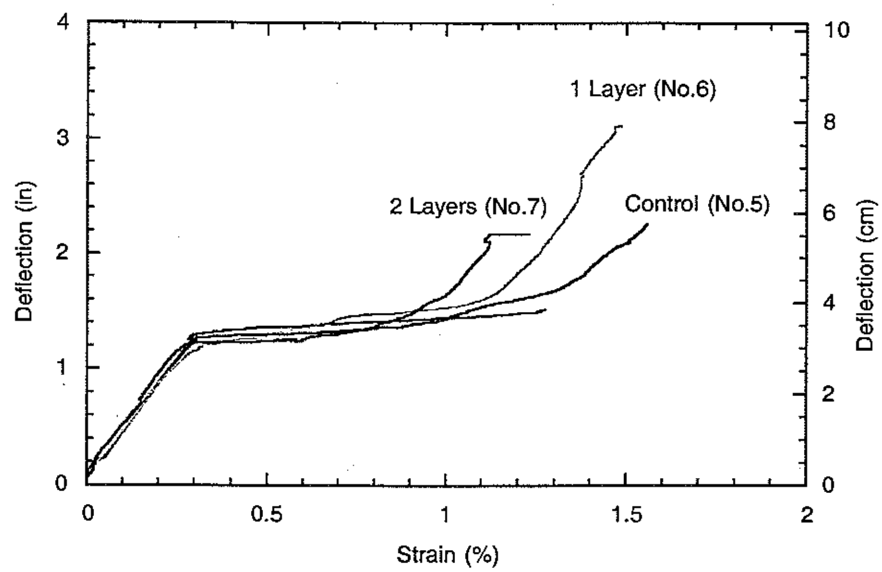


Figure 20 Deflection-strain curves of reinforcing bar of beams with reinforcement ratio, $0.54\rho_{max}$, at two strengthening levels.

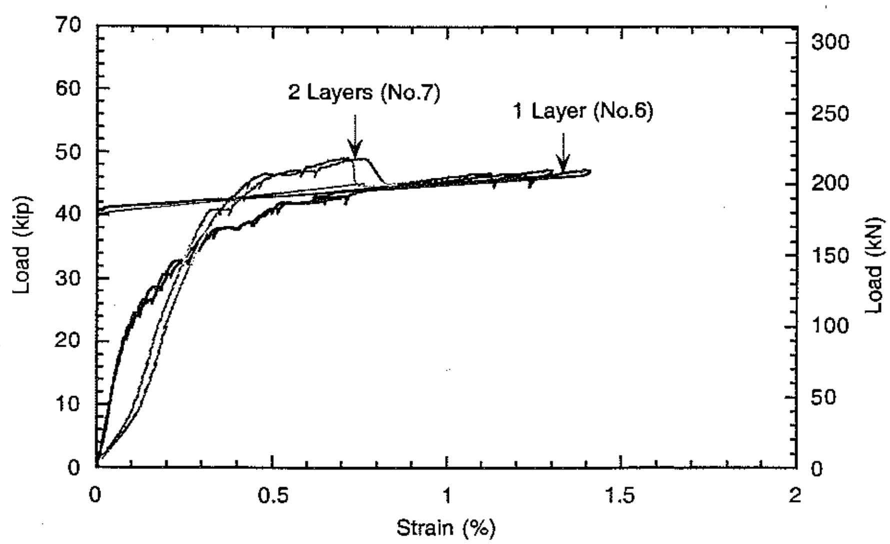


Figure 21 Load-strain curves of CFRP sheet of beams with reinforcement ratio, $0.54\rho_{max}$, at two strengthening levels.

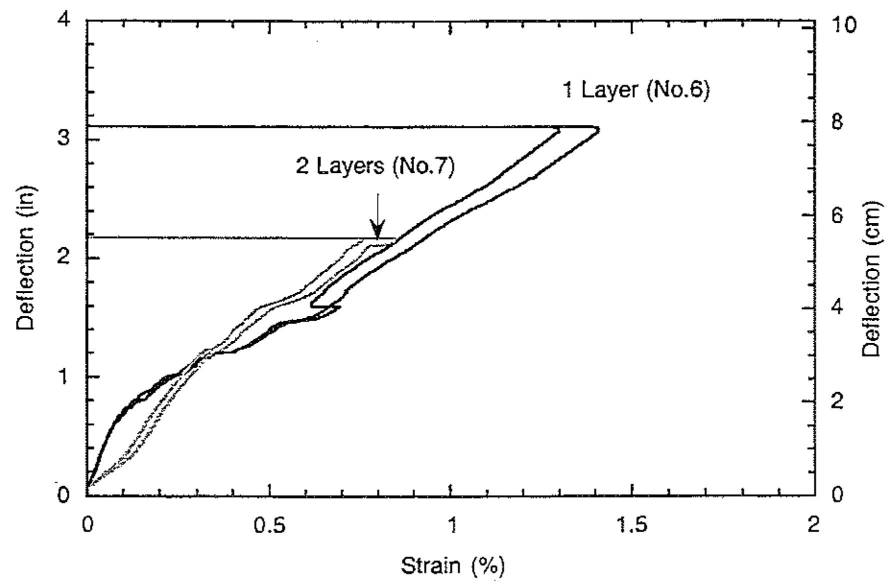


Figure 22 Deflection-strain curves of FRP sheet of beams with reinforcement ratio, $0.54\rho_{max}$, at two strengthening levels.

3.1.3 Influence of Strengthening Level

In the experimental study of the parameters, steel reinforcement ratio and strengthening level, several findings were observed. Some numerical results are given in Table 7.

1. For a given reinforcement ratio, the ultimate load capacity increases with the strengthening level, or the number of CFRP sheets (Figure 23).
2. The increment in ultimate load increase obtained by strengthening was almost proportional to the strengthening level or number of CFRP sheets (Figure 24). However, this direct relationship should be further confirmed experimentally in beams with higher strengthening levels and higher reinforcement ratios.
3. The almost equal slopes of the two lines shown in Figure 24 suggest that the increment of ultimate load achieved by strengthening is not significantly affected by the reinforcement ratio. However, the lower the reinforcement ratio, the higher the strengthening effect in terms of percent increase in ultimate load capacity (Figure 25).

4. The ultimate deflection of strengthened beams decreased as the strengthening level increased (Figure 26), that is a lower ductility is obtained. This is one of the disadvantages of beams strengthened using CFRP sheets. However, the strengthened beams had, after failure or delamination of the CFRP sheets, a minimum loading capacity and ductility which were almost same as those of the control beam, in spite of the fact that the concrete cover in the constant moment zone was severely damaged (see Section 3.7 Residual Strength of Beam after Failure).
5. The total tensile force contributed by the CFRP sheets at failure, increased linearly with the strengthening level (or number of sheets used) (Figure 27).

   In Figure 27, the total tensile force was calculated from the following equation:

$$T = \Delta M / (h - h_f/2)$$

   where ΔM is the difference in moment capacity (observed experimentally) between the strengthened beam and the control beam, taken at the failure deflection of the strengthened beam, h is the total depth of the beam and h_f is the depth of the flange of the T section used. Note that $(h - h_f/2)$ represents, as a first approximation, the lever arm from the centroid of the strengthening sheets to the centroid of the compression force in the concrete. In Figure 27, a value of calculated T is also shown; it is based on equilibrium and strain compatibility of the section at the observed ultimate load of the beam. The two values are comparable and provide some confidence in the calculations.

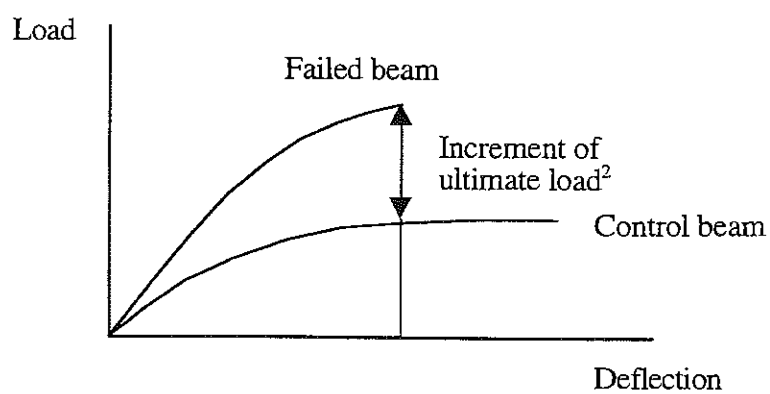
Table 7 Summary of test results of Beams with different strengthening levels.

Beam No.	1	2	3	4	5	6	7
Reinforcement ratio	$0.27\rho_{max}$	$0.27\rho_{max}$	$0.27\rho_{max}$	$0.27\rho_{max}$	$0.54\rho_{max}$	$0.54\rho_{max}$	$0.54\rho_{max}$
No. of CFRP sheet layer	0	1	2	4	0	1	2
Type of failure[1]	S.Y	T.F.	I.S.F	I.S.F	S.Y	I.S.F	I.S.F
Ultimate load, kN	114.8	135.0	140.4	160.7	188.0	209.9	222.0
Ultimate deflection, mm	16.4	8.3	5.6	4.9	8.8	7.7	5.2
Yielding load, kN	89	102	116	133	180	191	207
Increment of ultimate load, kN	0	20.2	25.6	45.9	0	21.9	34.0
Increment of yielding load, kN	0	13.3	26.7	44.5	0	11.1	26.7
Increment of ultimate load[2], kN	0	34.7	44.9	66.3	0	23.1	36.5
Measured CFRP tensile stress, MPa	–	3285	2074	1544	–	3049	1654
Measured CFRP tensile force, kN	–	55.2	69.4	103.6	–	40.9	55.6
Calculated CFRP tensile force, kN	–	56.9	73.4	108.5	–	37.8	59.6
Measured shear stress of concrete (MPa)	–	0.76	0.96	1.43	–	0.57	0.80
Calculated shear stress of concrete (MPa)	–	0.79	1.02	1.50	–	0.52	0.82

1: S.Y: Steel Yielding; T.F. = Tensile failure of CFRP sheet

I.S.F: Interfacial Shear Failure of concrete (delamination failure)

2: Increment of the ultimate load at the same deflection of failed beam, see sketch below.



6. It seems that the contribution of the shear resistance of concrete to the strength of the interface, linearly increases with the strengthening level (Figure 28). In Figure 28, the interface shear stress of concrete was calculated based on the assumption of equal shear stress along the shear span.

   The shear stress at the interface between the CFRP sheets and the concrete at failure of the beam (by delamination) was calculated from the tensile force T (described above in 5.) assuming horizontal shear stresses are equally distributed over the shear span. This leads to the equation:

$$\tau = T / (WL_a)$$

   Where W is the width of the CFRP sheet or contact area, and L_a is the distance from the loading point (maximum moment point) to the end of the CFRP sheet. Here also the second value of shear "calculated" and shown in Figure 28 corresponds to the value of T obtained from analysis of the section at ultimate moment capacity as described in 5.

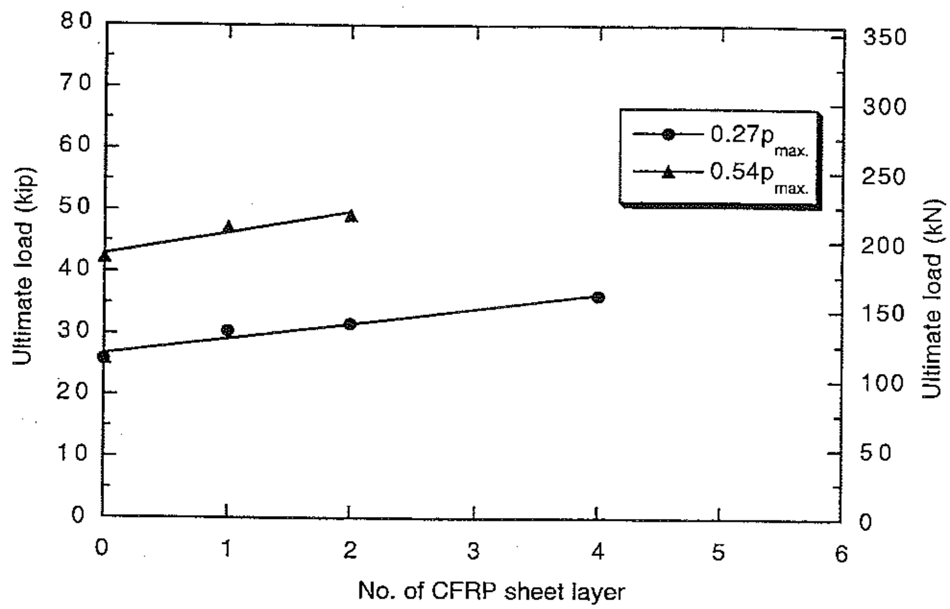


Figure 23 Relationship between ultimate load and strengthening level.

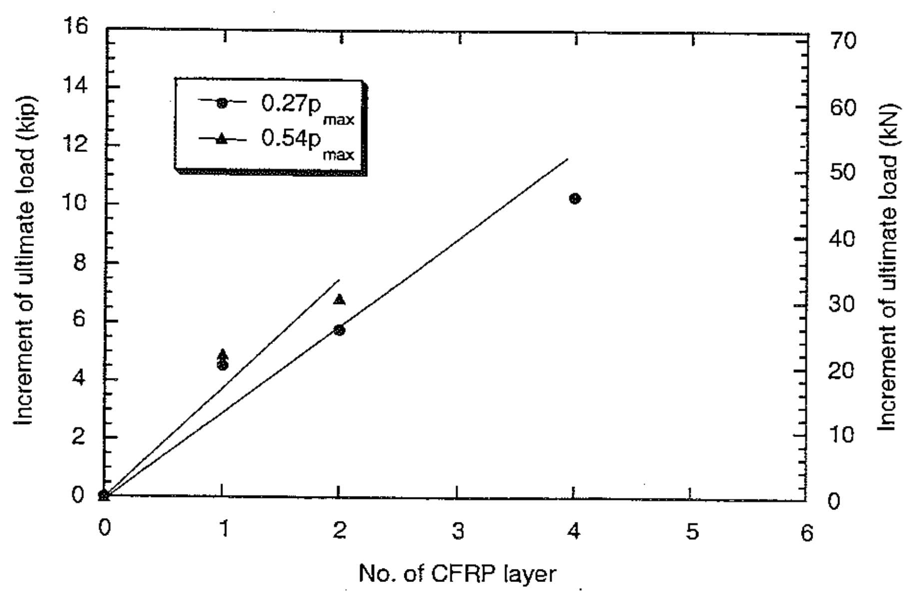


Figure 24 Increment of ultimate load versus strengthening level.

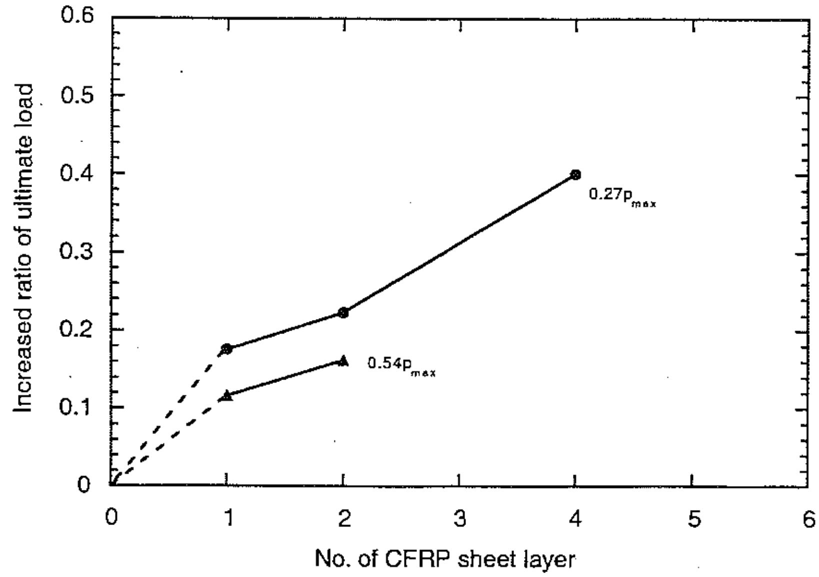


Figure 25 Increase in ratio of ultimate load versus strengthening level.

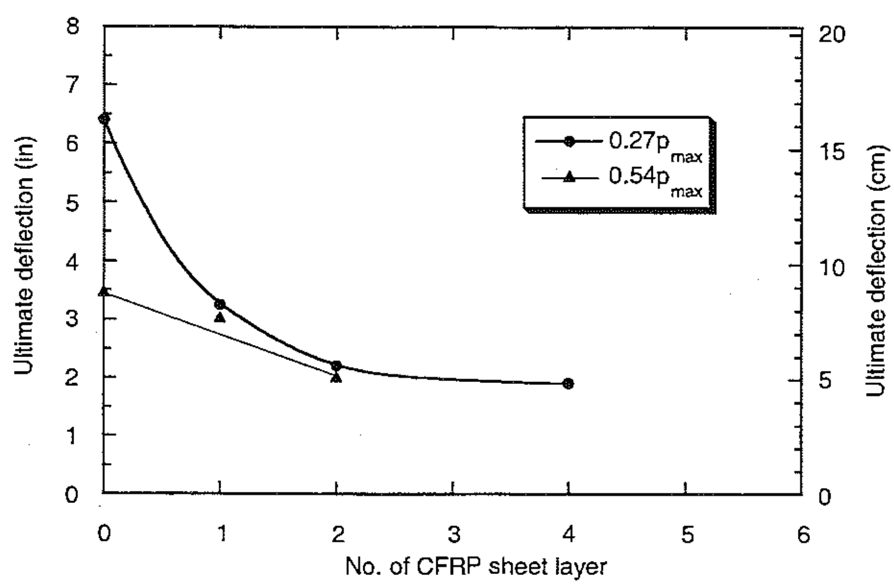


Figure 26 Ultimate deflection versus strengthening level.

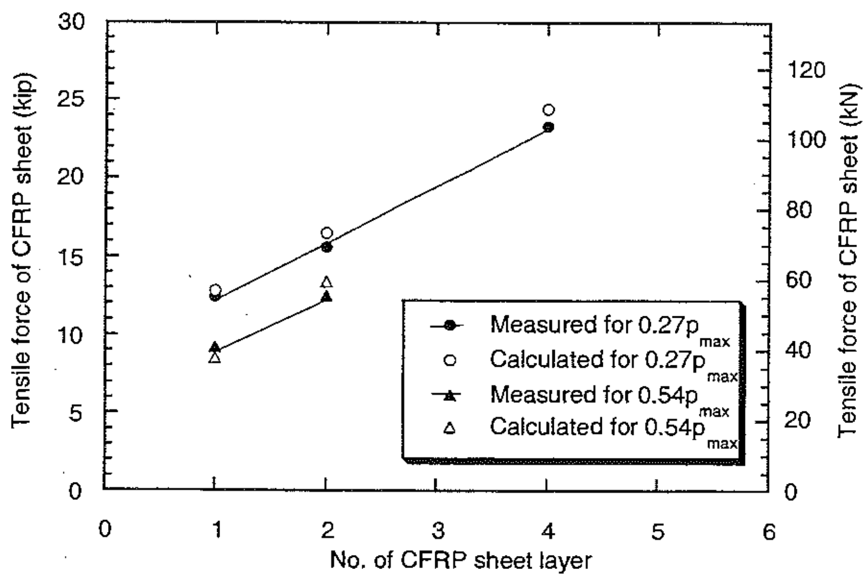


Figure 27 Tensile force of CFRP sheet versus strengthening level.

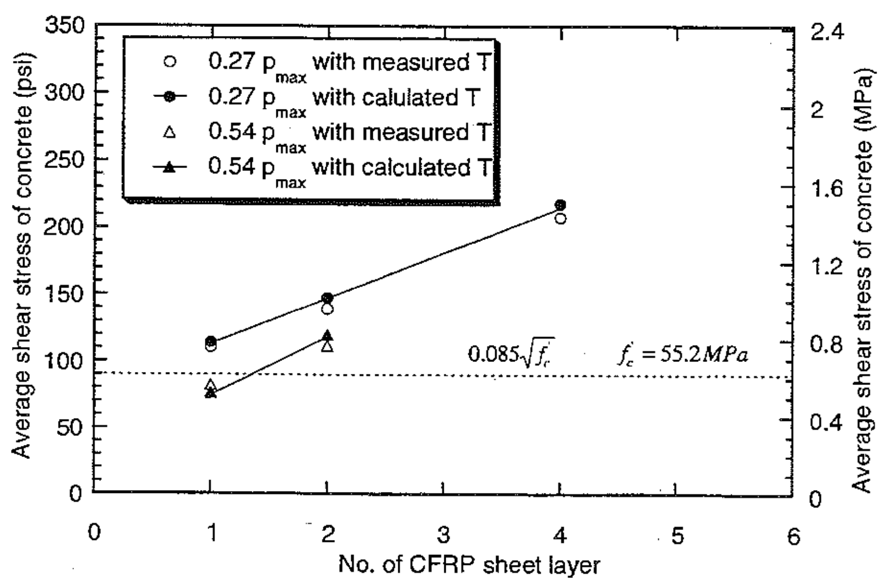


Figure 28 Shear stress of concrete at delamination of CFRP sheet.

3.2 Influence of Strengthening System

To compare the two strengthening systems tested, the results of Beams No. 8 and No. 8-1 are compared with those of Beam No. 7. Beams No. 8 and No. 8-1 were strengthened using CarboDur (Sika) CFRP plate having a tensile strength of 2,400 MPa and a tensile modulus of 150 GPa, while Beam No. 7 was strengthened with Forca Tow sheet (Tonen), having a tensile strength of 3,480 MPa and a tensile modulus of 228 GPa. One layer of Sika CFRP plate and Tonen CFRP sheet, 100 mm wide, provided a tensile strength of 2,868 N/mm width and 574 N/mm width, respectively. The 40 mm wide Sika CFRP plate used in Beam No. 8 was equivalent to two layers of 100 mm wide Tonen CFRP sheets in terms of failure strength.

All three beams failed by interfacial shear failure in the concrete (delamination) just above the epoxy adhesive. Unlike Beam No.7, Beam No. 8 had no pieces of concrete cover spalled off (Figure 29). This fact can be attributed to the smaller bond width of 40 mm of the Sika plate. Beam No. 8 was in very good shape even after delamination of the CFRP plate, and was later re-used as Beam No. 8-1. strengthened with 100 mm wide Sika plate thus the strengthening level of Beam No. 8-1 was about 2.5 times that of Beam No. 8. Upon testing, Beam No. 8-1 was severely damaged by the delamination failure as shown in Figure 30, where large chunks of concrete cover spalled off.

Figure 31 compares the load-deflection curves of the three strengthened beams, using Sika and Tonen systems, with the control beam. It can be observed that the load-deflection response of Beam No. 8 using the Sika system was almost the same as Beam No. 7 with the Tonen system; the difference in the tensile modulus of the two systems (the modulus of Sika CFRP plate was about two thirds that of Tonen CFRP sheet) had little influence on the elastic portion of the curve. The ultimate load and deflection of the beam with the Sika system were respectively about 4% and 11% less than those of the beam with the Tonen system. Beam No. 8-1 with the 100 mm Sika plate showed a very stiff load-deflection response because of its high strengthening level. The ultimate strength of Beam No. 8-1 was higher while its ultimate deflection was smaller than those of Beams No.7 and No. 8 (Figure 31).

Figures 32 to 37 show the load-strain and deflection-strain curves of respectively the reinforcing bar, concrete in top flange, and the CFRP laminate at the mid span section. These figures are provided as additional information. For instance, it can be observed that the strain in the Sika CFRP plate in Beam No. 8

was almost the same as that of the Tonen CFRP sheet in Beam No. 7 (Figures 36 and 37).

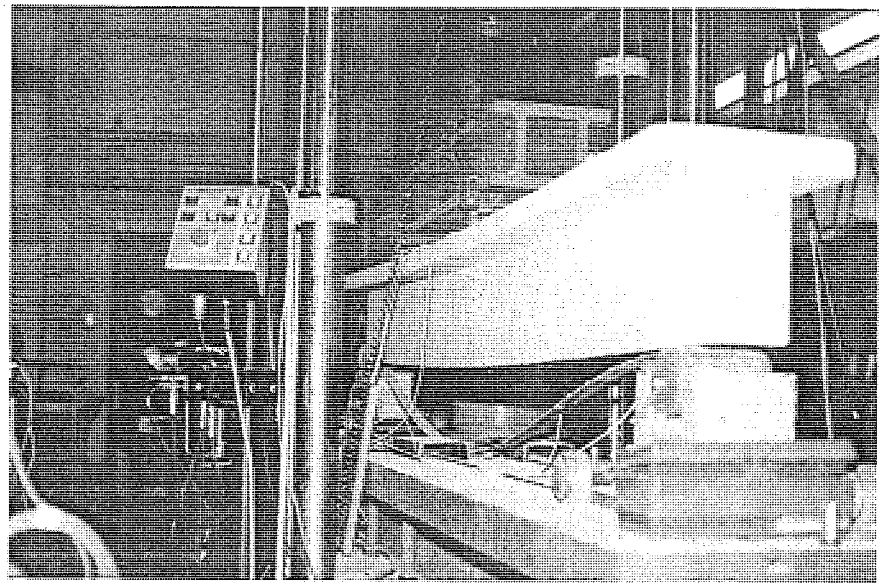


Figure 29 Interfacial shear failure (delamination) of concrete in Beam No.8

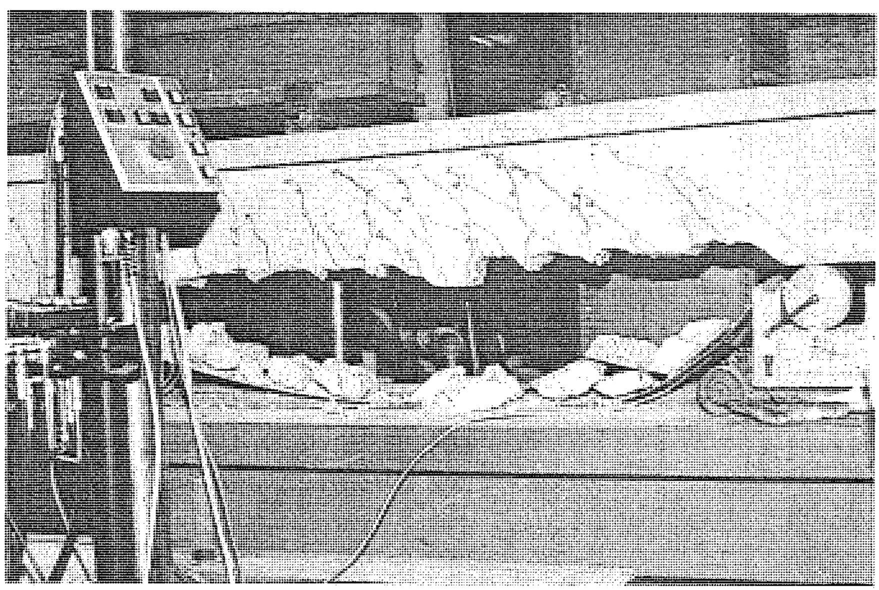


Figure 30 Interfacial shear failure of concrete in Beam No.8-1

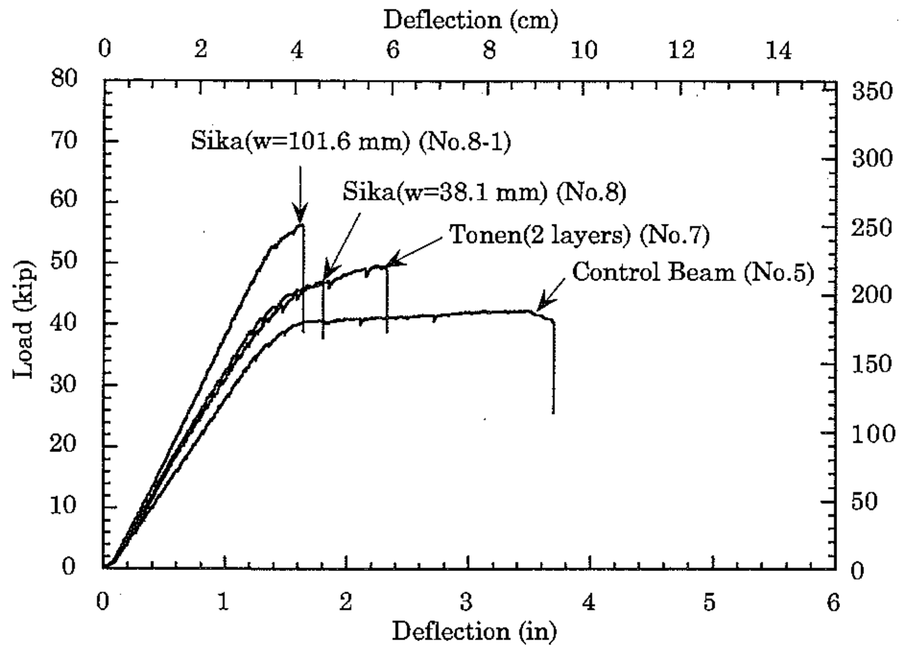


Figure 31 Load-deflection curves for different strengthening systems.

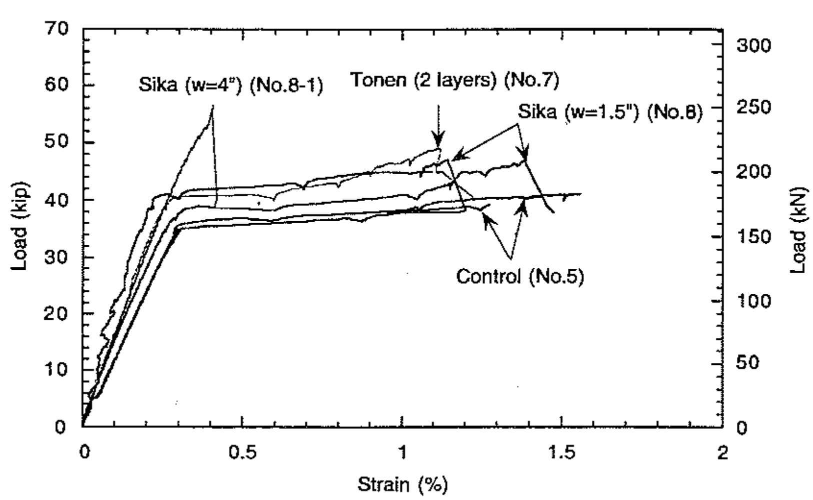


Figure 32 Load-strain curves of reinforcing bar.

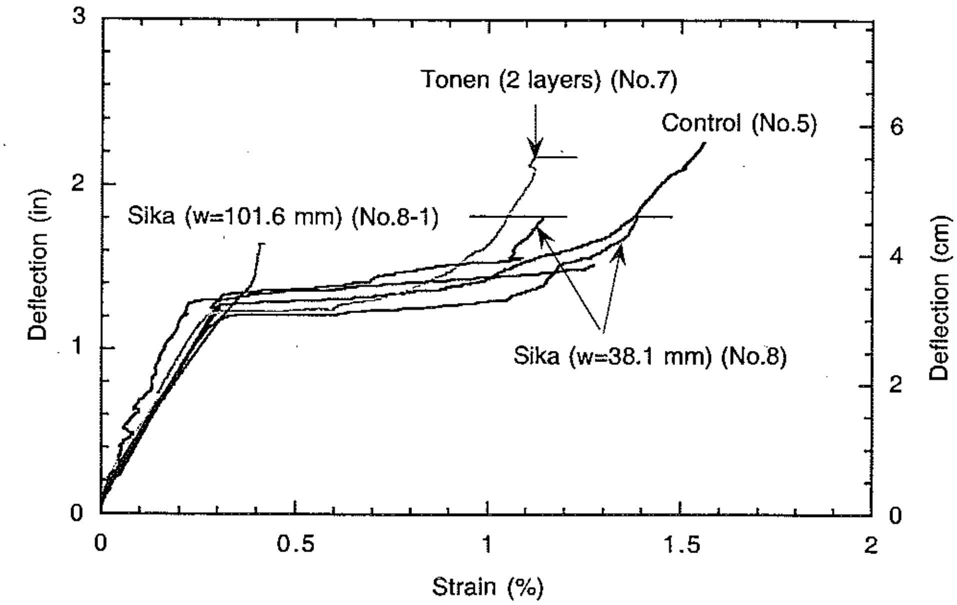


Figure 33 Deflection-strain curves of reinforcing bar.

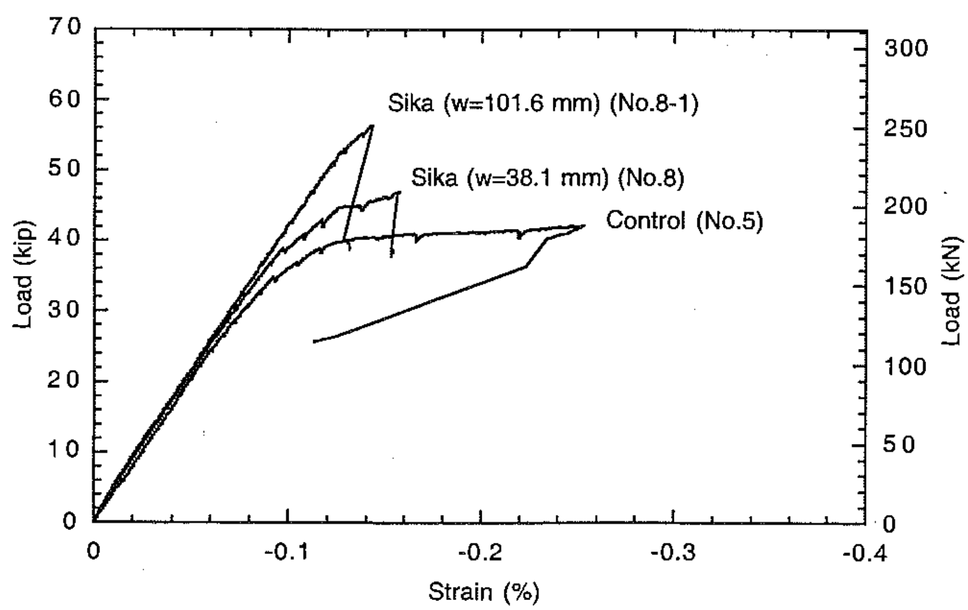


Figure 34 Load-strain curves of concrete.

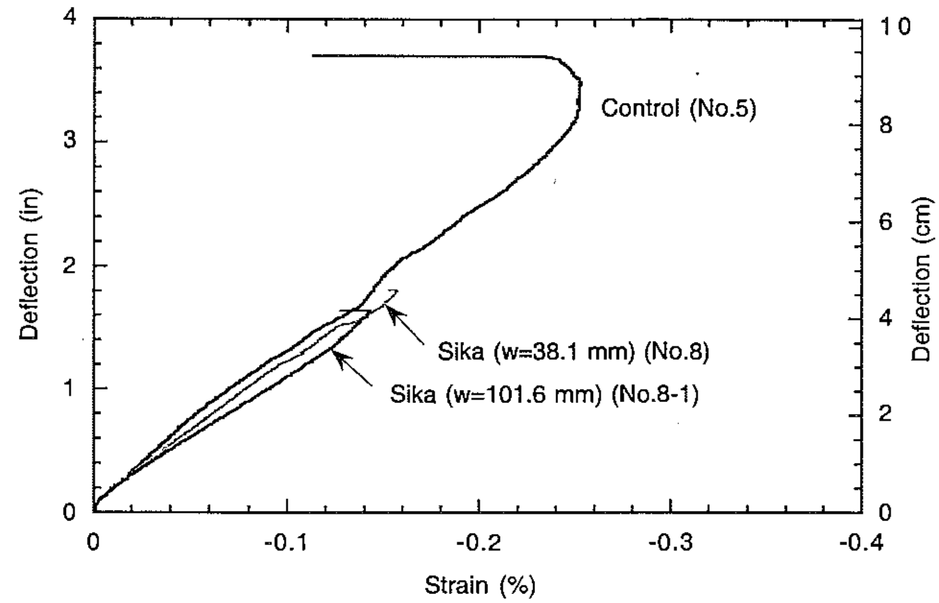


Figure 35 Deflection-strain curves of concrete.

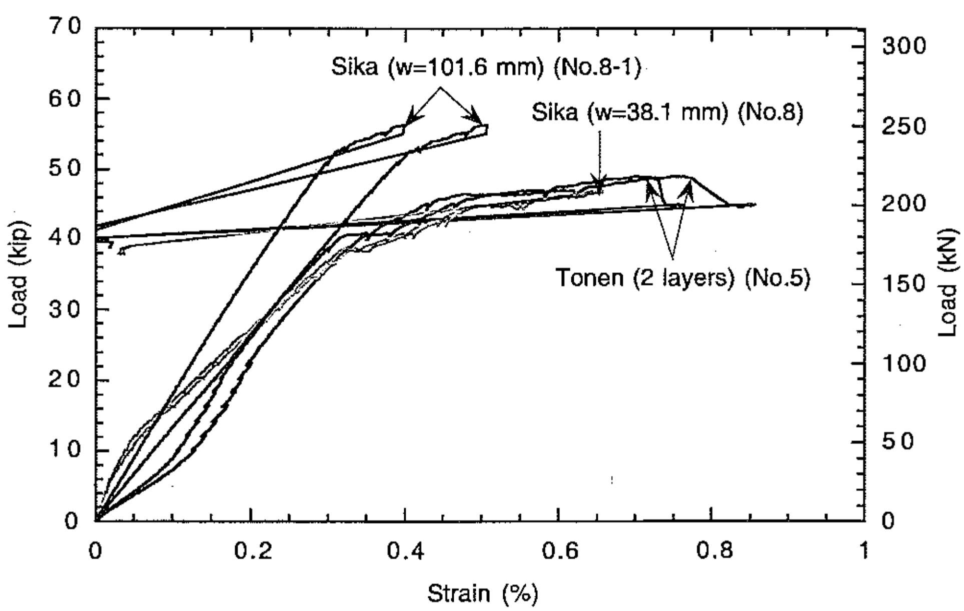


Figure 36 Load-strain curves of CFRP sheet.

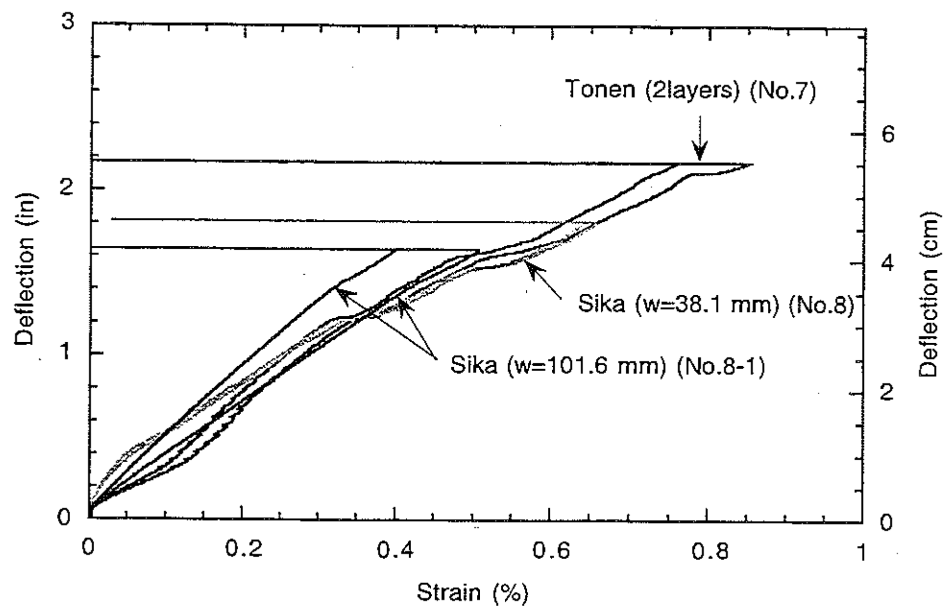


Figure 37 Deflection-strain curves of CFRP sheet.

3.3 Influence of Concrete Cover

To investigate the effect of concrete cover, Beams No. 12 and No. 9 were tested and compared to Beam No. 7. Beams No. 12 and No. 7 had 25 mm and 50 mm net concrete cover depth, respectively. Beam No. 9 had 25 mm existing concrete cover as cast, and 25 mm additional cover added with epoxy mortar to simulate a repair. All beams had the same effective depth from the top concrete fiber to the longitudinal steel reinforcement and the same reinforcement ratio, $.54\rho_{max}$.

Beam No. 12 with 25 mm deep cover and Beam No. 7 with 50 mm deep cover failed by interfacial shear failure of concrete (delamination). Beam No. 9 with repaired concrete cover, failed by interfacial bond failure between the repair mortar and the existing concrete (Figure 38)

The load-deflection curves of the three beams are compared in Figure 39. Surprisingly, it is observed that the concrete cover considerably affected the ultimate deflection of the strengthened beam. Beam No. 12 with 25 mm cover had about 35% larger ultimate deflection than Beam No. 7 with 50 mm cover. Yet they both had about the same strength.

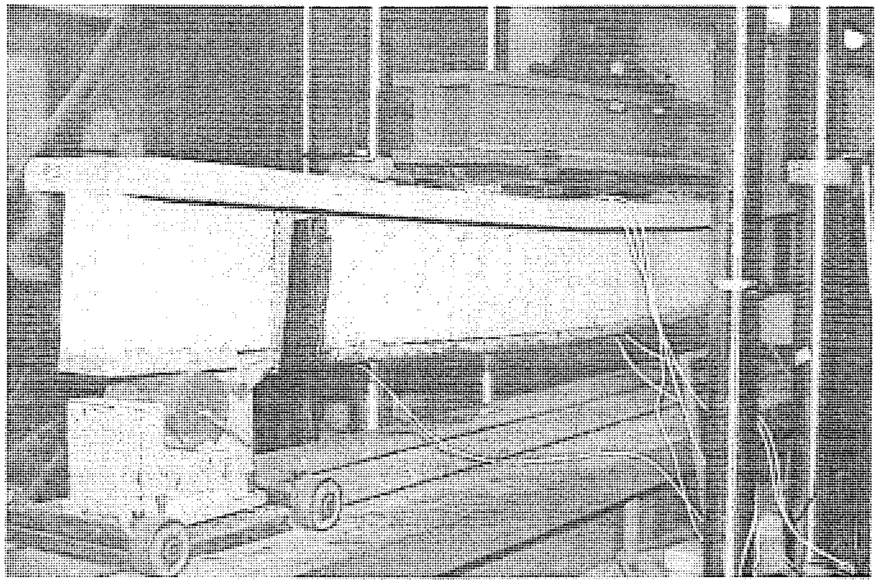


Figure 38 Interfacial bond failure of CFRP sheet in Beam No. 9.

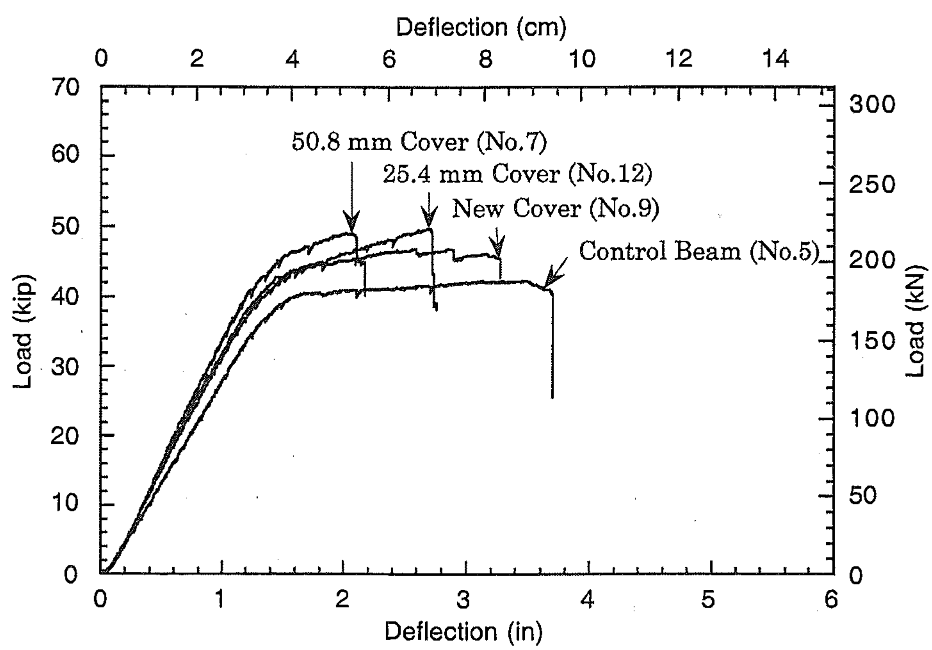


Figure 39 Load-deflection curves of beams with different concrete cover.

Compared to Beams No. 7 and 12, Beam No. 9 showed a more ductile behavior. Its ultimate deflection, 83 mm, was about 60% larger than that of Beam No. 7. The reason for this increase is believed to be due to the gradual debonding of the interface between the repair mortar and existing concrete. However, the increase in load due to strengthening, 20.2 kN, was about 35% less than that of Beam. No. 7.

3.4 Influence of End Anchorage

To evaluate the effect of different anchorage systems, Beams No. 10 and No. 14 were tested and compared with Beam No. 7. For Beam No. 10, CFRP sheet was extended and glued up to the support without having a wrapped end U-shaped anchorage. Beam No. 14 had neither a wrapped end anchorage nor an extended end anchorage. Beam No. 7 had a 100 mm wide wrapped end anchorage.

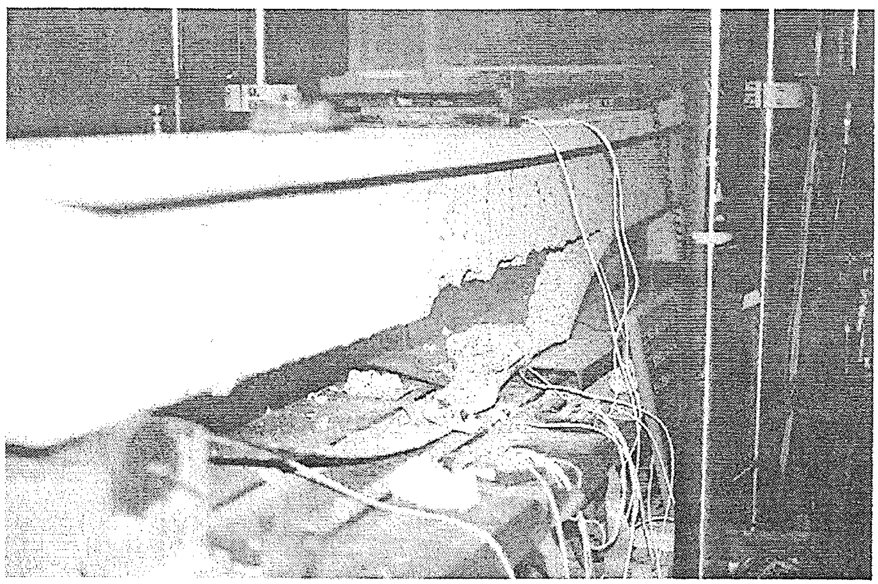


Figure 40 Interfacial shear failure of concrete in Beam No. 10.

All three beams failed by interfacial shear failure of concrete (delamination) regardless of their type of anchorage. In all three beams, delamination damaged the concrete cover in the constant moment zone (Figures 40 and 41). Beam No. 14 had one piece of concrete cover torn off at the end of the CFRP sheet.

The load-deflection curves of the three beams are compared in Figure 42. For all practical purposes, all three beams had the same ultimate load and the same ultimate deflection, regardless of their anchorage system.

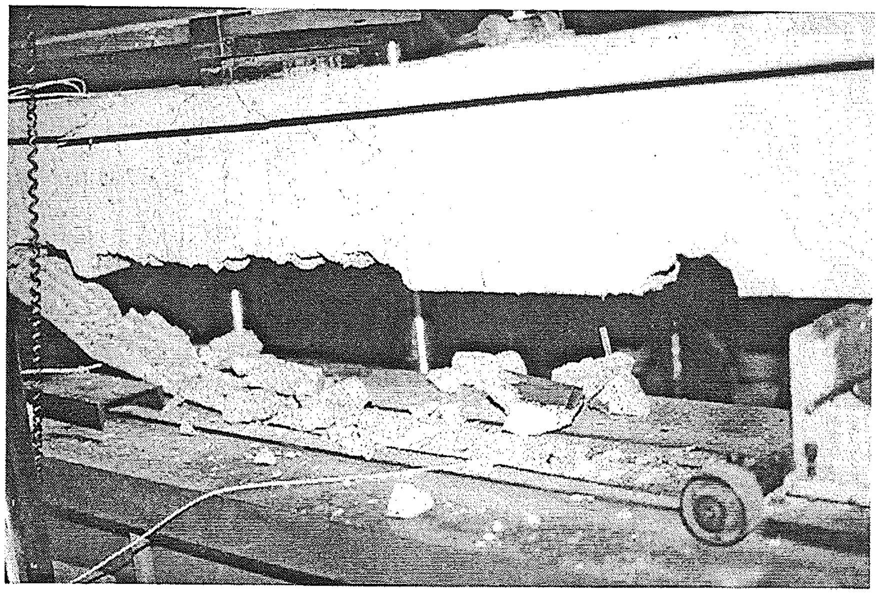


Figure 41 Interfacial shear failure of concrete in Beam No. 14.

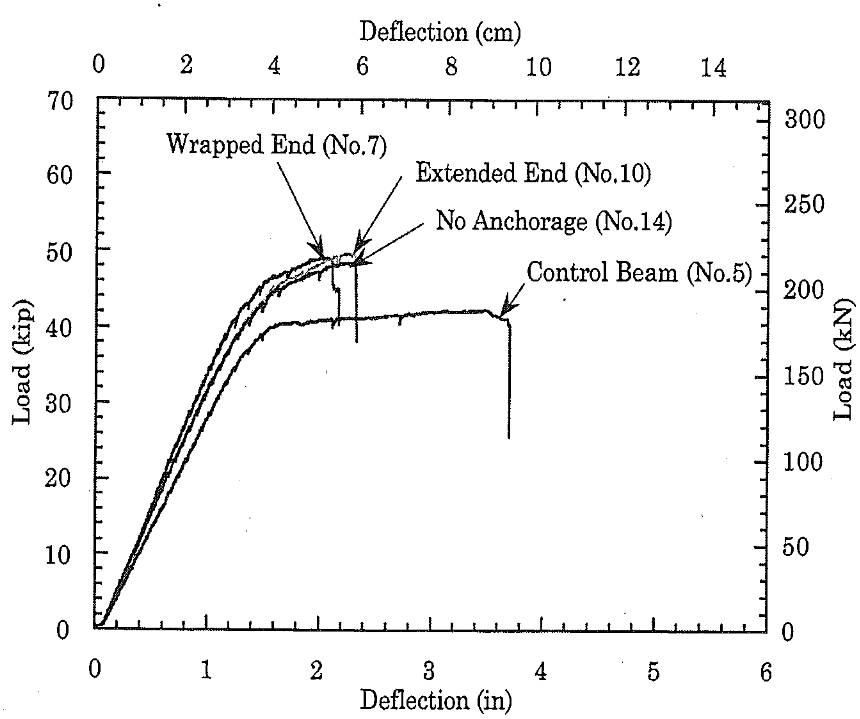


Figure 42 Load-deflection curves of beams with different anchorage.

This may suggest that in spite of stress concentration effects at the ends of the glued-on sheets, the failure crack may have started elsewhere along the beam, very likely at the site of an existing flexural crack, near the point of maximum load. So improving the end anchorage beyond what is strictly needed as development length, does not seem necessary.

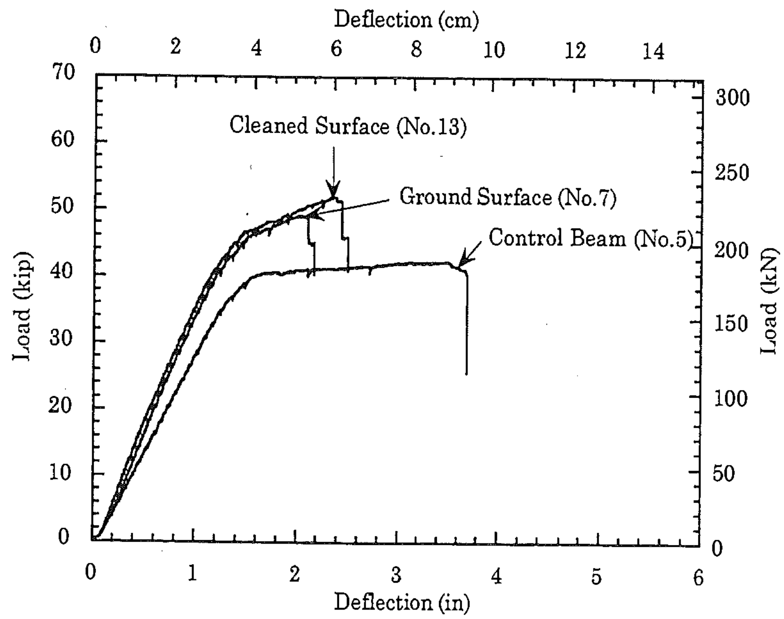


Figure 43 Load-deflection curves of beams with different surface preparation.

3.5 Influence of Surface Preparation

To evaluate the importance of surface preparation, Beam No. 13 was prepared without grinding the surface of concrete to be glued-on. The concrete surface was simply vacuum cleaned and wiped off with clean cloth. The load-deflection response curve of Beam No. 13 is compared in Figure 43 with that of Beam No. 7 which was prepared with a ground surface. No notable difference can be observed. Both beams failed by interfacial shear failure of the concrete (delamination). In fact, Beam No. 13 without ground surface, showed a slightly higher strength and deflection. This test result may imply that a well cleaned surface of concrete is enough to develop a good bond strength with epoxy adhesive. However, this conclusion should be experimentally confirmed by additional tests, because the concrete used in this test was new, and its surface was no subjected to any prior environmental deterioration.

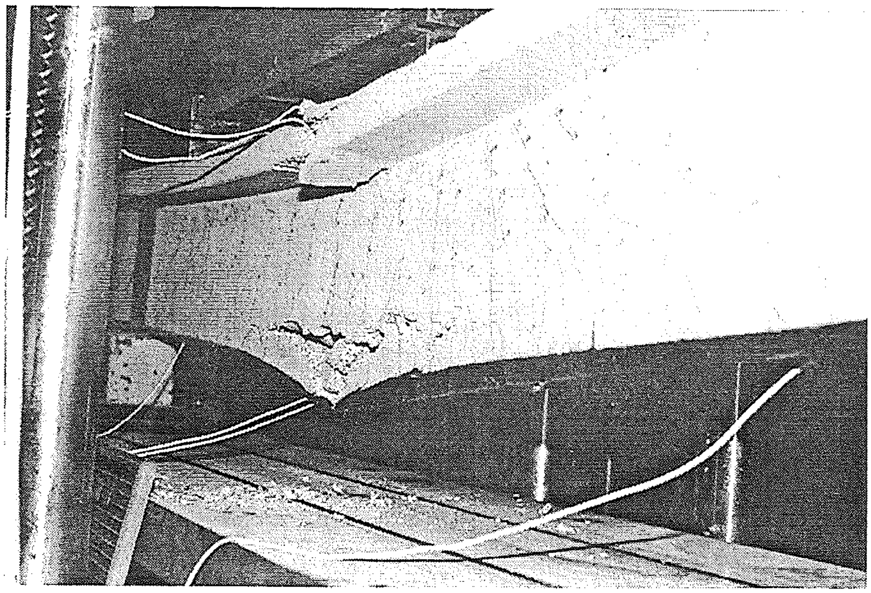


Figure 44 Inter-laminar shear failure of Beam No. 11.

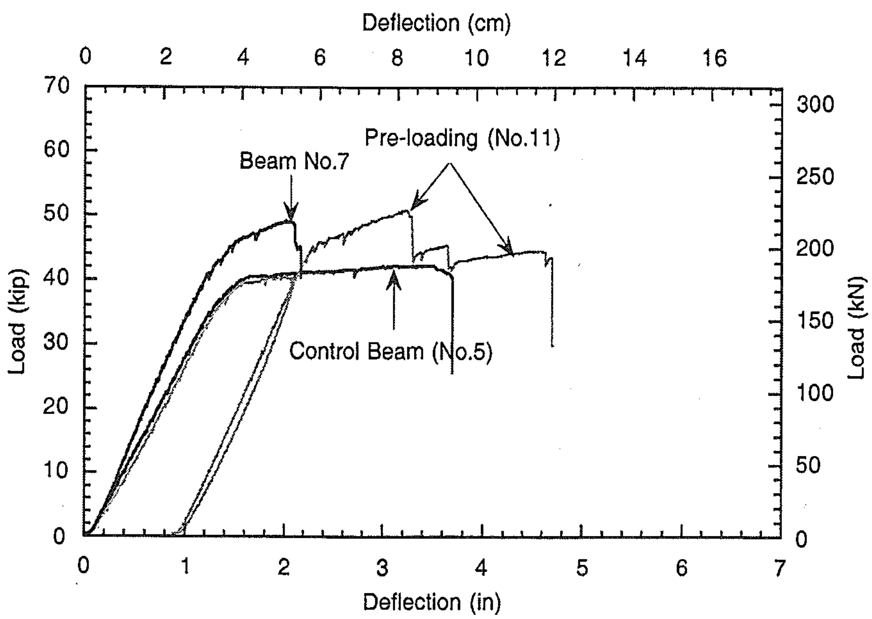


Figure 45 Load-deflection curves of beams with different loading history.

3.6 Influence of Loading History

To investigate the effect of loading history, before the application of CFRP sheets, on the flexural behavior of strengthened beams, Beam No. 11 was pre-loaded to about 180 kN, a load clearly beyond the yielding of the steel reinforcing bars (Figures. 44 and 45). The permanent deflection and maximum crack width in the pre-cracked beam at unloading, were about 24 mm and 0.9 mm, respectively. After unloading, two layers of Tonen CFRP sheets were glued to the beam, following exactly the same procedure as for previous beams. The reason for pre-loading and pre-cracking was to simulate a severely damaged beam in real situation.

Unlike other beams, the pre-loaded beam, Beam No. 11 failed by inter-laminar shear failure within the first layer of the glued on CFRP sheet (Figure 44). This failure mode is thought to be due to the imperfect penetration of epoxy into the fibers of the first layer, and is independent of the pre-loading condition. At the time of gluing the CFRP sheet, the epoxy resin was six-month old (after the bucket was first opened). The resin was quite thick and very difficult to mix because of hardened lumps and high viscosity. However, in spite of that, the ultimate load of Beam No. 11 was about the same as that of Beam No.7. Also, its concrete cover in the constant moment zone, separated from the reinforcing bars like the beams that failed by interfacial shear failure of concrete. These two observations suggest that the inter-laminar shear failure in the CFRP sheet occurred just before the interfacial shear of concrete was reached.

Figure 45 shows that the load-deflection curve of Beam No. 11 is close to being bilinear while that of Beam No. 7 is curvilinear. This is because Beam No. 11 was pre-loaded and pre-cracked beyond yielding, prior to application of the CFRP sheet. Note that after inter-laminar shear failure, Beam No. 11 achieved the same loading capacity as the control beam, that is until the beam failed by compression failure of concrete in the top flange.

From these test results, it can be concluded that pre-loading and pre-cracking beyond reinforcement yielding have no serious influence on strengthening effect. Therefore, the CFRP glued-on sheet strengthening technique can be applied for flexural strengthening even to severely damaged beams.

3.7 Residual Strength of Beam after Failure

After failing by interfacial shear failure of concrete, and in order to check the residual strength, Beam No. 13 was unloaded and reloaded without any repair until the beam failed by compression failure of concrete. As shown in Figure 46, Beam No. 13 was severely damaged by delamination failure. The concrete cover in the constant moment zone was completely separated, and the two reinforcing bars in the lower layer were exposed to air. The bars had already yielded at time of delamination failure.

Figure 47 shows that Beam No. 13 attained the same ultimate load as the control beam even after severe damage by delamination. This fact is due to the two reinforcing bars in the lower layer which resisted the applied load because they had good bond in the shear span, even though they had lost bond in the constant moment zone. This result can be used as to provide a minimum safety level for strengthened beams, after failure of their CFRP sheet or plate.

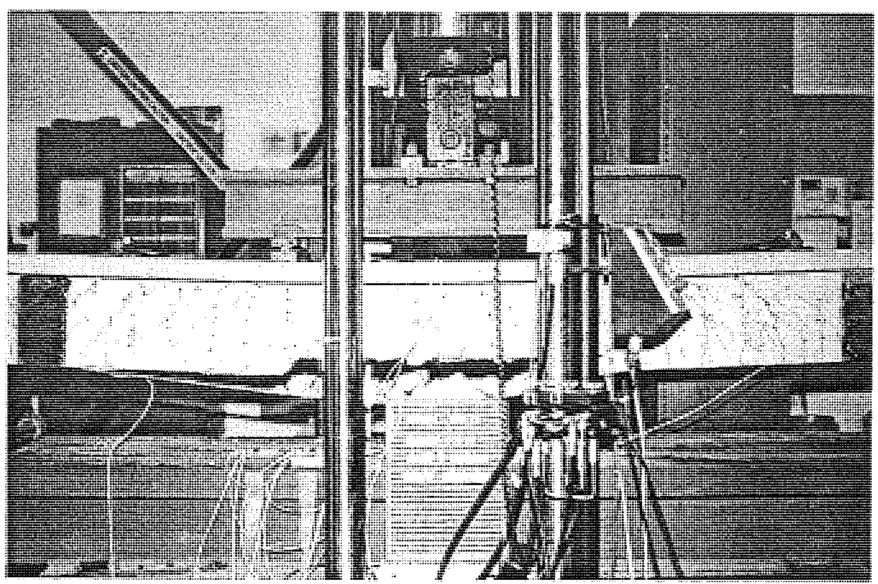


Figure 46 Condition of Beam No. 13 failed by delamination before re-testing.

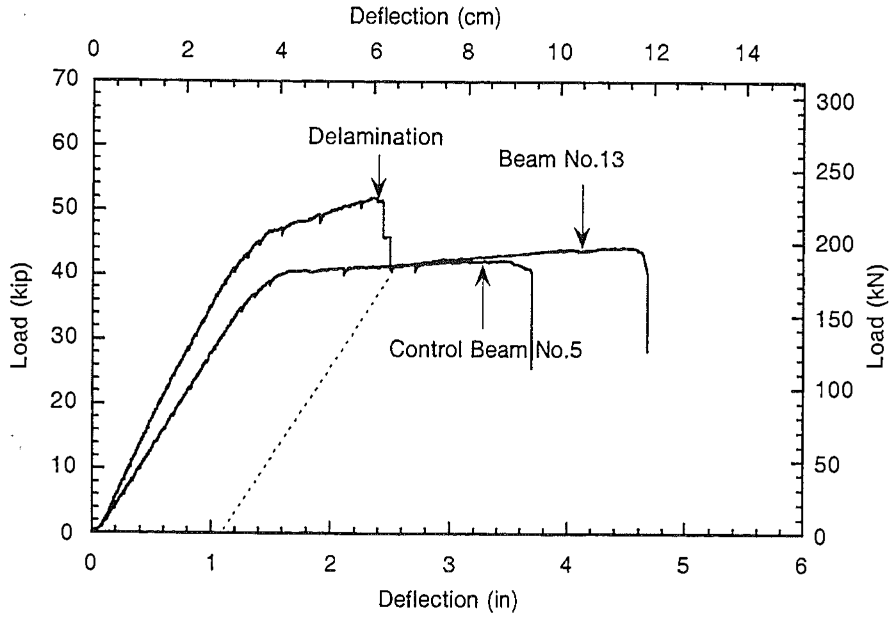


Figure 47 Load-deflection curves of Beam No. 13.

4. CONCLUSIONS

This investigation dealt with the flexural behavior of reinforced concrete beams strengthened using glued-on carbon fiber reinforced plastic (CFRP) sheets or plates. Based on the observation and analysis of the experimental test results the following conclusions can be drawn.

1) The strengthening technique using externally bonded CFRP sheets or plates can significantly improve the ultimate loading capacity of reinforced concrete beams; however, their ultimate deflection is reduced. On the other hand, the strengthened beams had, after failure or delamination of the CFRP sheets, a minimum loading capacity and ductility which were almost same as those of the control beam.

2) Strengthening with CFRP sheets can inhibit the growth of large cracks by helping distribute a large number of smaller cracks; it also protect the steel reinforcement from further corrosion.

3) In general, normally strengthened beams fail by interfacial shear failure (delamination) within the concrete, instead of by tensile failure of the CFRP sheet or plate.

4) In normally strengthened beams, the increment in ultimate load obtained by strengthening was almost proportional to the strengthening level or number of CFRP sheets. However, this direct relationship should be further confirmed experimentally in beams with higher strengthening levels and higher reinforcement ratios.

5) For a given reinforcement ratio, the ultimate load capacity increases with the strengthening level, or the number of CFRP sheets. However, the steel reinforcement ratio of the reinforced concrete beam to be strengthened, does not seem to have a significant effect on the increment of load at ultimate achieved by strengthening. This implies that the lower the reinforcement ratio, the higher the strengthening effect in terms of percent increase in ultimate load capacity.

6) The ultimate deflection of strengthened beams decreased in comparison to the control beam as the strengthening level increased, thus leading to a lower ductility. This is one of the disadvantages of beams strengthened using CFRP sheets. However, the strengthened beams had, after failure or delamination of the CFRP sheets, a minimum loading capacity and ductility which were almost same as those of the control beam.

7) Beams using the strengthening system with CFRP plate (Sika system) showed the same load versus deflection response as beams using the strengthening system with CFRP sheet (Tonen system), even though the tensile modulus of the CFRP plate was two thirds that of the CFRP sheet. In this investigation where non-trained students were involved, it was found that the system using CFRP plate is easier and more convenient for flexural strengthening than that using the CFRP sheet.

8) The strengthened beam with a smaller concrete cover had slightly higher ultimate load and considerably larger ultimate deflection than the control beam with normal concrete cover.

9) The beam strengthened after having a repaired concrete cover failed by gradual interlaminar debonding at the interface between existing concrete and repair mortar; it led to a ductile behavior, but did not achieve an adequate level of strengthening.

10) Using a U-shaped end anchorage of the CFRP sheet did not help attain higher ultimate loads or deflections, in comparison to having no anchorage. However, extending the sheet up to the supports led to slightly higher ultimate load and deflection. Therefore, the extended end anchorage system is recommended because it is easier to apply.

11) Preparing the concrete surface by grinding prior to the application of CFRP sheets was not more effective than simply vacuum cleaning and wiping the surface. However this conclusion should be further confirmed in real beams with deteriorated concrete surfaces.

12) Pre-loading and pre-cracking a beam beyond reinforcement yielding had no serious influence on the strengthening effect. Therefore, the CFRP glued-on strengthening technique can be applied even to severely damaged beams.

13) The beam that failed by CFRP sheet delamination and was damaged due to severe concrete cover spalling had, upon reloading, the same ultimate load and deflection as the control beam, even though the damage was severe. This fact can insure some minimum safety level for beams strengthened using CFRP sheets, should failure by delamination or tension of the sheet occur.

14) Based on the limited number of tests carried out, it seems that the contribution of the shear resistance of concrete to the strength of the interface linearly increases with the strengthening level. For this conclusion, the interface shear stress of concrete was calculated based on the assumption of equal shear stress along the shear span.

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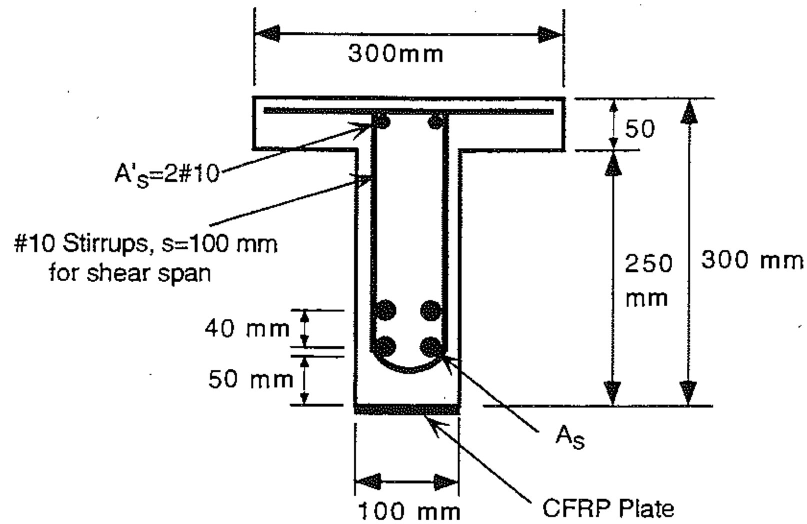
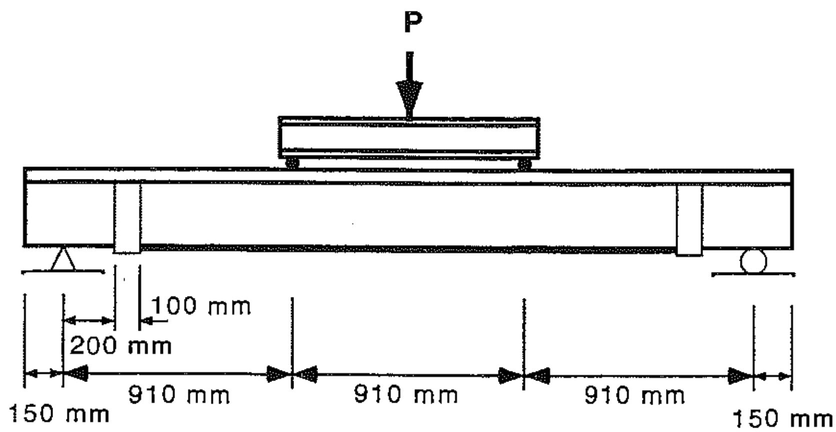
6. APPENDIX A

Moment Capacity Calculations





- Distance from centroid of steel to top layer of concrete

For beam No.1-4:

d1=(12-2-4/8*1/2)*25.4 = 248 mm, d2=(12-2-4/8*1/2-1.5)*25.4 = 210 mm

de = (9.75*0.2*2+8.25*0.11*2)/0.62*25.4 = 235 mm

For beam No. 5-14:

d1=(12-2-5/8*1/2)*25.4 = 246 mm, d2=(12-2-5/8*1/2-15)*25.4 = 208 mm

de = (d1+d2)/2 = 227 mm

- Concrete compressive strength = 55.2 MPa , β1 = 0.65

- Steel yield strength:

For bars #10 &#13, f_y = (73*0.11+62*0.2)/0.31*6.895 = 455 MPa

For #16 bar, f_y = 455 MPa

- As balance

T-section behavior: As balance = Concrete force due to equilibrium(Cc) /f_y

For de = 227 mm, cd (neutral axis) for balance condition = 134 mm> flange height (hf = 51 mm). Find Cc for T-section:

Cc = (b-bw)*hf*0.85*f'c + 0.85*f'c*bw*β1*Cd

Cc = 900 kN

As balance = 900,000/455 = 1977 mm^2 , As_{max} = 0.75 As balance = 1483 mm^2

For de = 235 mm, cd (neutral axis) for balance condition = 139 mm> flange height (51 mm). Find Cc for T-section:

Cc = (b-bw)*hf*0.85*f'c + 0.85*f'c*bw*β1*Cd

Cc = 914 kN

As balance = 914,000/455 = 2000 mm^2 , As_{max} = 0.75 As balance = 1506 mm^2

Take **As_{max} = 1490 mm^2**

- M_{max}

M_{max} = As_{max}*f_y*(de-a/2), where a=As_{max}*f_y/(0.85*f'c*bf) if a>hf T-section behavior. This equation can not be applied.

For de = 227 mm, a = 47 mm < 51 mm, Rectangular (R) beam behavior,
M_{max} = 138 kN-m

For de = 235 mm, a = 47 mm < 51 mm, R-behavior, M_{max} = 143 kN-m

- M_n

M_n = As*f_y*(de-a/2), where a=As*f_y/(0.85*f'c*bf) if a>hf T-section behavior. This equation can not be applied.

For As = 400 mm^2, de = 235 mm, a = 13 mm < 51 mm, R-behavior! M_n = 42 kN-m

For As = 800 mm^2, de = 227 mm, a = 25 mm < 51 mm, R-behavior! M_n = 78 kN-m

- M_{As}, M_{FRP}

For beam No. 1-4:

<u>1 layer CFRP</u>

$T_{As} = 0.62*66*4.448 = 182$ kN

$T_{FRP} = 3.28*4*4.448 = 58.36$ kN

$T_{total} = 240.55$ kN

$a = T_{total} / (0.85*f'c*bf) = 16.8$ mm

$M_{As} = T_{As}*(de-a/2) = 41.09$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 17.30$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 58.39$ kN-m

<u>2 layers CFRP</u>

$T_{As} = 0.62*66*4.448 = 182$ kN

$T_{FRP} = 3.28*8*4.448 = 116.7$ kN

$T_{total} = 298.73$ kN

$a = T_{total} / (0.85*f'c*bf) = 20.8$ mm

$M_{As} = T_{As}*(de-a/2) = 40.73$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 34.35$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 75.08$ kN-m

<u>4 layers CFRP</u>

$T_{As} = 0.62*66*4.448 = 182$ kN

$T_{FRP} = 3.28*16*4.448 = 233.43$ kN

$T_{total} = 415.44$ kN

$a = T_{total} / (0.85*f'c*bf) = 29$ mm

$M_{As} = T_{As}*(de-a/2) = 40$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 67.77$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 107.77$ kN-m

For beam No. 5-14:

<u>1 layer CFRP</u>

$T_{As} = 1.24*66*4.448 = 364$ kN

$T_{FRP} = 3.28*4*4.448 = 58.36$ kN

$T_{total} = 128.74$ kN

$a = T_{total} / (0.85*f'c*bf) = 29.5$ mm

$M_{As} = T_{As}*(de-a/2) = 77.30$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 16.93$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 94.24$ kN-m

<u>2 layers CFRP</u>

$T_{As} = 1.24*66*4.448 = 364$ kN

$T_{FRP} = 3.28*4*4.448 = 116.7$ kN

$T_{total} = 480.74$ kN

$a = T_{total} / (0.85*f'c*bf) = 33.5$ mm

$M_{As} = T_{As}*(de-a/2) = 76.59$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 33.62$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 110.18$ kN-m

<u>25 mm cover &2 CFRP layers</u>

$M_{As} = T_{As}*(de-a/2) = 76.59$ kN-m

$M_{FRP} = T_{FRP}*(h-25-a/2) = 30.65$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 107.21$ kN-m

<u>Sika. b= 38 mm</u>

$T_{As} = 364$ kN

$T_{FRP} = 109.15$ kN

$T_{total} = 473.18$ kN

$a = T_{total} / (0.85*f'c*bf) = 33$ mm

$M_{As} = T_{As}*(de-a/2) = 76.65$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 31.47$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 108.12$ kN-m

<u>Sika. b=102 mm</u>

$T_{As} = 364$ kN

$T_{FRP} = 291.08$ kN

$T_{total} = 655.10$ kN

$a = T_{total} / (0.85*f'c*bf) = 45.7$ mm

$M_{As} = T_{As}*(de-a/2) = 74.06$ kN-m

$M_{FRP} = T_{FRP}*(h-a/2) = 82.06$ kN-m

$M_{(As+FRP)} = M_{As} + M_{FRP} = 156.13$ kN-m

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UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 4: Behavior of Beams Strengthened For Shear

by

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16. Abstract

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow-sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:
Volume 1 – Summary Report
Volume 2 – Literature Review
Volume 3 – Behavior of Beams Strengthened for Bending
Volume 4 – Behavior of Beams Strengthened for Shear
Volume 5 – Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications

The part of the investigation dealing with reinforced concrete beams strengthened in shear is described in this volume (volume 4), where the results are also analyzed, compared, and discussed. The experimental program comprised three rectangular concrete beams and three T-beams. The test parameters included two different shear-span ratios. Two commercially available strengthening systems were tested, the Sika CFRP plate system (CarboDur), and the Tonen CFRP sheet system. Both systems were used for shear strengthening and for two specimens they were also used for shear and bending strengthening. Other selective parameters investigated included two levels of longitudinal steel reinforcement ratio before strengthening and two steel shear reinforcement levels. Conclusions are drawn and some recommendations for design are suggested.

17. Keywords bond, carbon fiber, composites materials, laminates, rehabilitation, reinforced concrete, repair, strengthening systems		18. Distribution Statement No restrictions. This document is available to the public through the Michigan Department of Transportation	
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The opinions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors.

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PREFACE

This project titled: *"Repair and Strengthening of Reinforced Concrete Beams using CFRP Laminates"* is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of the project were:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) glued-on plates for repair and strengthening of concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

The project consisted of 8 tasks as follows:

- A report containing a literature review and a comprehensive synthesis of the latest state of knowledge on the glued -on FRP technique (Task 1);
- Laboratory testing and verification of the selected CFRP glued-on technique according to the proposed experimental program: bending (Task 2), shear (Task 3), freeze-thaw (Task 4), temperature and high cyclic amplitude load (Task 5);
- An interim and final report summarizing the experimental results (Task 6). The interim report will cover the bending and freeze-thaw tests;
- A summary of field specifications and "how to" details for implementation in field applications;
- Guidelines for design based on the experience developed from the experimental work (Task 7);
- Field monitoring of application of the technique to one bridge selected by MDOT (Task 8a);
- Bridge testing before and after application of the glued-on plate (Task 8b to be conducted by professor A. Nowak, U of M)

This report summarizes the experimental program of beams strengthened for shear as per Task 3.

ABSTRACT

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow-sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report is part of a larger investigation on the use of carbon fiber reinforced plastic (CFRP) sheets for repair and strengthening of reinforced and prestressed concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, 2) to synthesize existing knowledge, 3) to identify optimum parameters for successful implementation, 4) to develop procedures for implementation in the field, and 5) to adapt the technique to the specific conditions encountered in the state of Michigan.

The experimental program includes four main parts: 1) tests of RC beams strengthened in bending; 2) tests of RC beams strengthened in shear; 3) freeze-thaw tests of strengthened beams followed by their test in bending; and 4) tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading.

The part of the investigation dealing with reinforced concrete beams strengthened in shear is described in this report, where the results are also analyzed, compared, and discussed. The experimental program comprised three rectangular concrete beams and three T-beams. The test parameters included two different shear-span ratios. Two commercially available strengthening systems were tested, the Sika CFRP plate system (CarboDur), and the Tonen CFRP sheet system. Both systems were used for shear strengthening and for two specimens they were also used for shear and bending strengthening. Other selective parameters investigated included two levels of longitudinal steel reinforcement ratio before strengthening and two steel shear reinforcement levels. Conclusions are drawn and some recommendations for design are suggested.

1. GENERAL

The study described in this report is part of a larger investigation on the use of carbon fiber reinforced plastic (CFRP) sheets for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, 2) to synthesize existing knowledge and develop procedures for implementation in the field, and 3) to adapt the technique to the specific conditions encountered in the state of Michigan.

The experimental program includes: 1) tests of RC beam strengthened in bending; 2) tests of RC beams strengthened in shear; 3) freeze-thaw tests of strengthened beams followed by their test in bending; and 4) tests in bending and shear of strengthened beams under low temperature, -29 C and high amplitude cyclic loading. The part of the investigation dealing with reinforced concrete beams strengthened in shear is described in this report, where the results are also analyzed, compared, and discussed.

2. EXPERIMENTAL PROGRAM

The experimental shear program comprised three rectangular and three T section concrete beams. The loading arrangement and cross sectional dimensions are shown in Figures 1 and 2 for rectangular and T beams, respectively. The rectangular beams were 100 mm wide, 250 mm deep, and 1.32 m long. The T beams with 100 mm webs were 300 mm deep and 1.93 m long. Their flanges were 300 mm wide and 50 mm thick. In most cases, the clear concrete cover for the reinforcing steel was kept at 50 mm.

To investigate the shear behavior, a three point shear test set-up was used (Figure 1). The selected shear span-to-depth ratio was 2.5 for the rectangular beams and 3.5 for the T beams. The rectangular beams were made without steel stirrups and strengthened for shear using CFRP sheets or plates. The T beams were provided with a flexural and shear reinforcement ratio of $0.89\rho_{max}$

and 0.91 $A_{v\ (max)}$ respectively and strengthened for bending and shear using CFRP sheets or plates. $A_{v\ (max)}$ is the shear reinforcement required for a reinforced concrete beam to resist a load at midspan that is allowed with the maximum longitudinal reinforcement ratio, ρ_{max}. Throughout the experimental study, the types of failures were closely observed. Also, the applied load, corresponding deflection and strains of longitudinal reinforcing bar, stirrup, and CFRP sheet (or plate) for flexural and shear strengthening were measured.

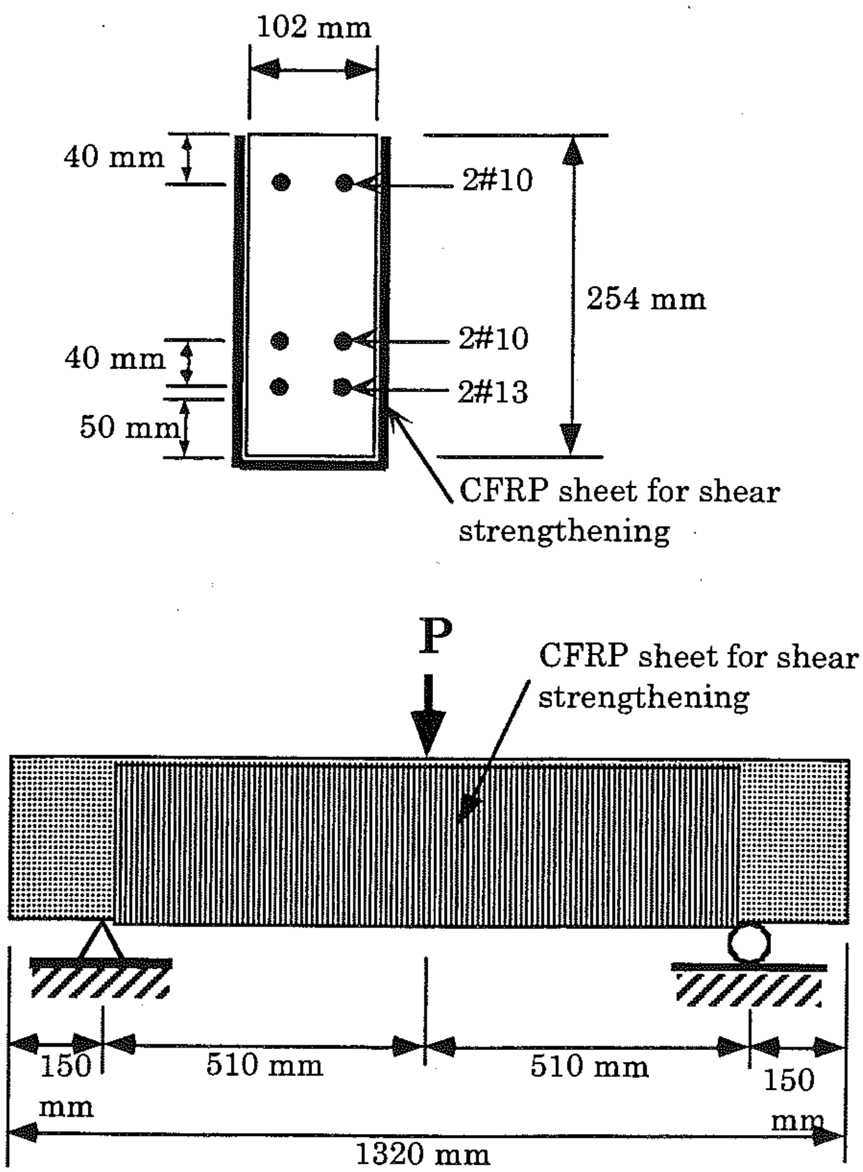


Figure 1 Typical cross section and loading arrangement for rectangular beams.

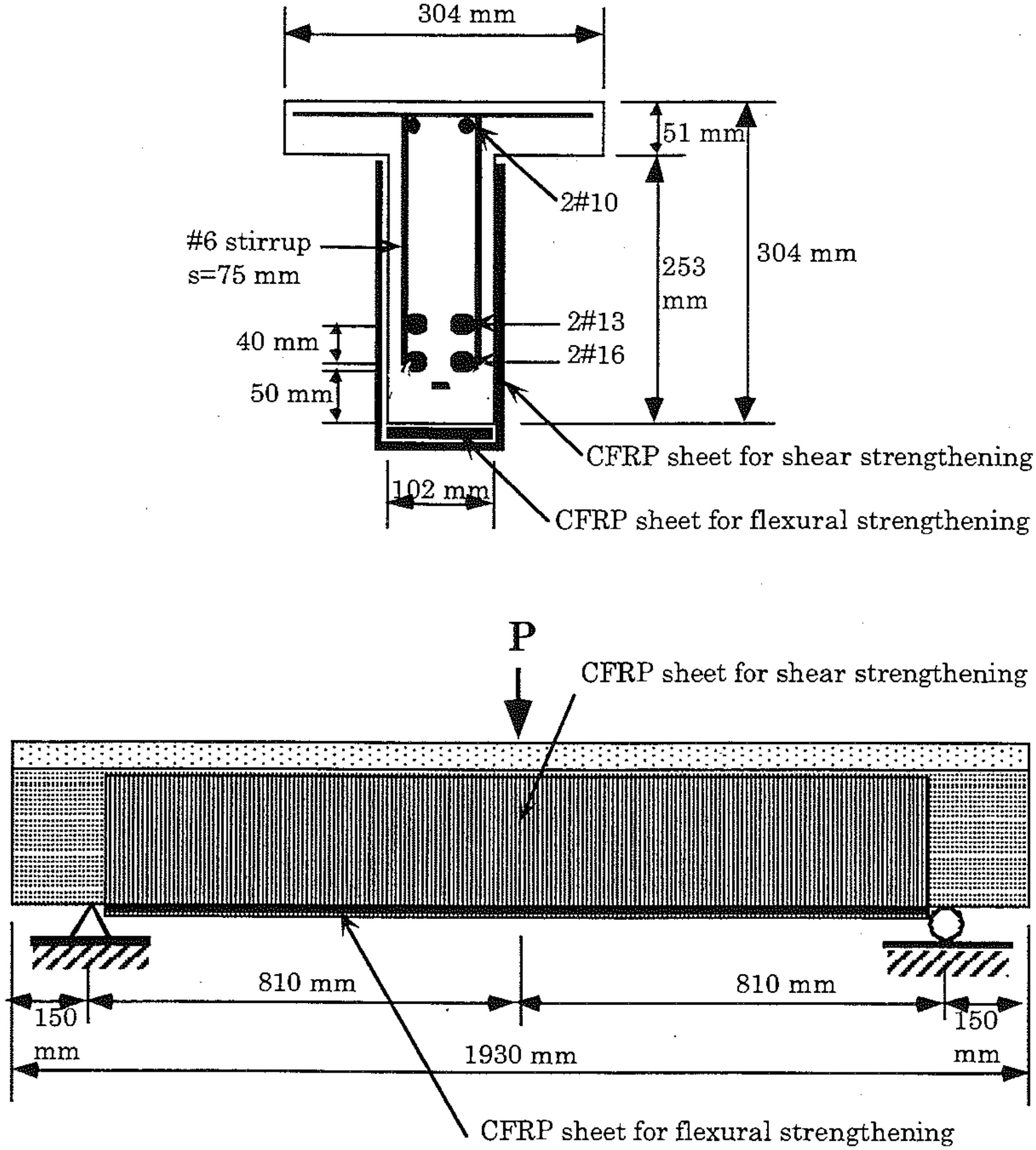


Figure 2 Typical cross-section and loading arrangement for T beams

2.1 Test Parameter

Several possible parameters for shear testing were proposed by the research team and evaluated by the Technical Advisory Group. Test parameters and experimental variables for the shear test are summarized in Table 1 and shown in Figures 3 & 4.

Initially, it was decided to use a high longitudinal reinforcement ratio in order to increase the ultimate bending resistance, be shear critical and observe shear strengthening effect. Accordingly, the longitudinal reinforcement ratio was designed to be approximately ρ_{max} for the rectangular beams and 0.5 ρ_{max} for the T-beams. These values were based on a target design compressive strength f'_c = 34.5 Mpa. However, the actual compressive strength of concrete (from the ready mix concrete company) was on the average 25.4 MPa. As a result the target values of reinforcement ratios were not achieved; actual values (summarized in Table 1) were significantly higher. It can be observed from Table 1 that the longitudinal reinforcement ratio of the rectangular beams was 1.41 ρ_{max} exceeding the balanced ratio, while for the T beams it was 0.89 ρ_{max} quite close to the maximum ratio, ρ_{max}, recommended by the AASHTO Code, as shown in appendix A. The rectangular beams did not have any steel stirrups. They were strengthened for shear using the CFRP sheets or plates. The T beams had steel stirrups reinforcement corresponding to 91% of the required area of stirrups for a reinforced concrete beam to resist a load at midspan that is allowed with a longitudinal reinforcement corresponding to ρ_{max} (maximum reinforcement ratio as recommended by the AASHTO Code). They were strengthened for shear to accommodate both the lack in shear resistance and the increase in bending resistance provided by the CFRP plate.

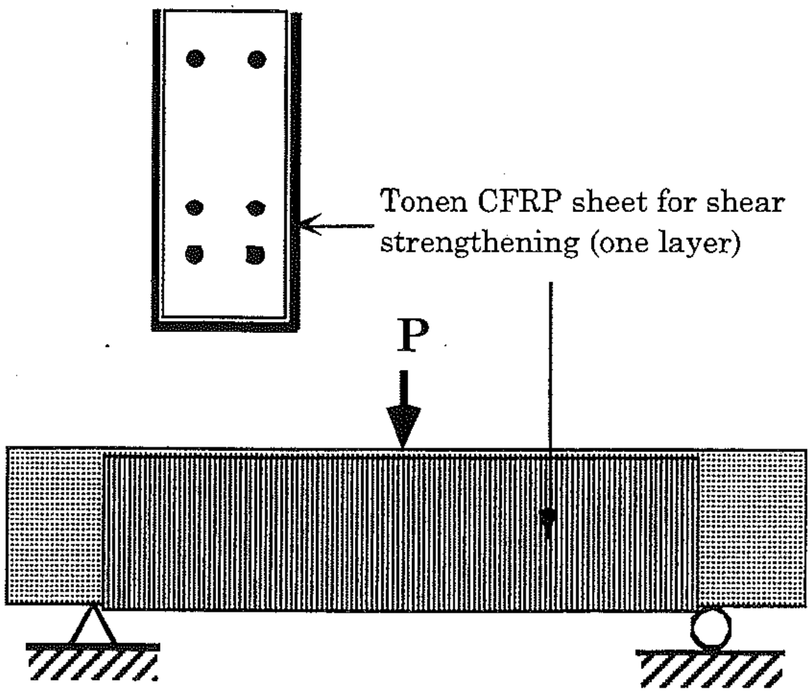
To investigate and evaluate the different strengthening systems, Tonen CFRP sheet (Forca Tow sheet) and Sika CFRP plate (CarboDur plate) were used for shear strengthening. For rectangular beams, one layer of Tonen CFRP sheet was glued on the web of Beam No. 2, and 25 mm wide Sika CFRP strips were glued on the web of Beam No. 3 at a spacing of 75 mm. The width and spacing of Sika CFRP strips were determined to be equivalent to the one continuous layer of Tonen CFRP sheet, based on equal tensile strength. The fiber direction of CFRP sheet or plate for shear strengthening was normal to the longitudinal direction of the beams.

Table 1 Test parameters and variables for shear test

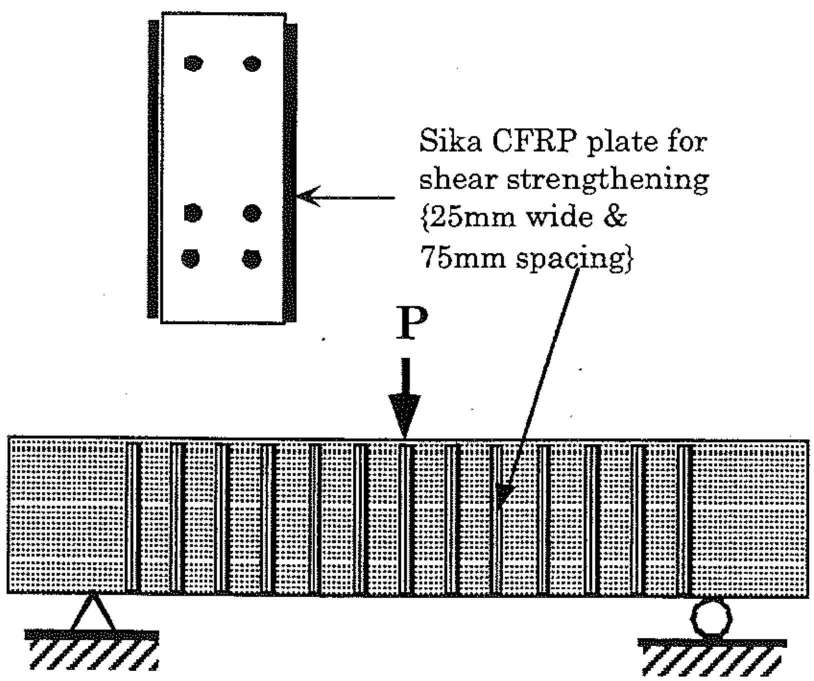
Beam No.	Section type	Shear reinforcement (A_v) Stirrup	Shear reinforcement (A_v) CFRP sheet	Shear span-to-depth ratio (a/d)	Long. Steel reinforcement ratio (ρ)	Strength-ening
1	102x254 Rect.-section	None	None	2.5	1.41 ρ_{max}	Control
2		None	$A_{v(FRP)}$ Tonen sheet	2.5	1.41 ρ_{max}	Shear
3		None	$A_{v(FRP)}$ Sika plate	2.5	1.41 ρ_{max}	Shear
4	304x304 T-section	0.91 $A_{v\,(max)}$	None	3.5	0.89 ρ_{max}	Control
5		0.91 $A_{v\,(max)}$	$A_{v(FRP)}$ Tonen sheet	3.5	0.89 ρ_{max}	Shear & Bending
6		0.91 $A_{v\,(max)}$	$A_{v(FRP)}$ Sika plate	3.5	0.89 ρ_{max}	Shear & Bending

Note: 1. Actual concrete compressive stress, f'c=25.4 MPa. Actual steel yield stress, fy=496 MPa and 483 MPa for rectangular and T sections

2. Maximum shear reinforcement ratio, $A_{v\,(max)}$, is for maximum longitudinal reinforcement ratio, ρ_{max}.

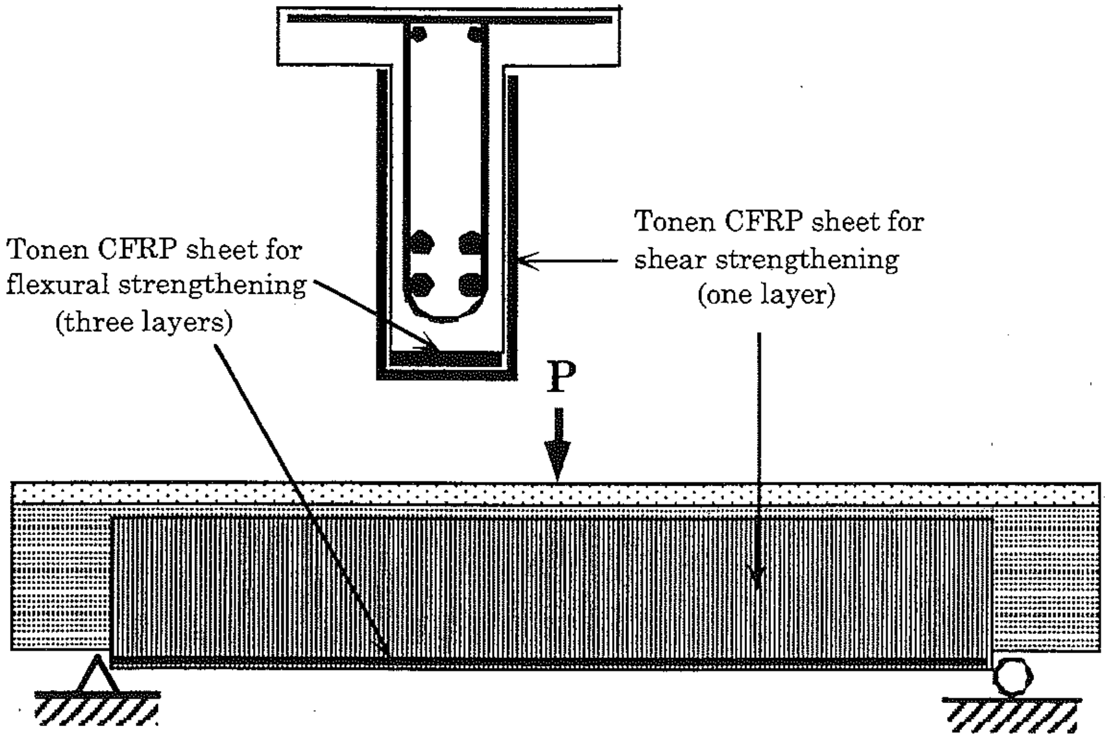


Beam No. 2

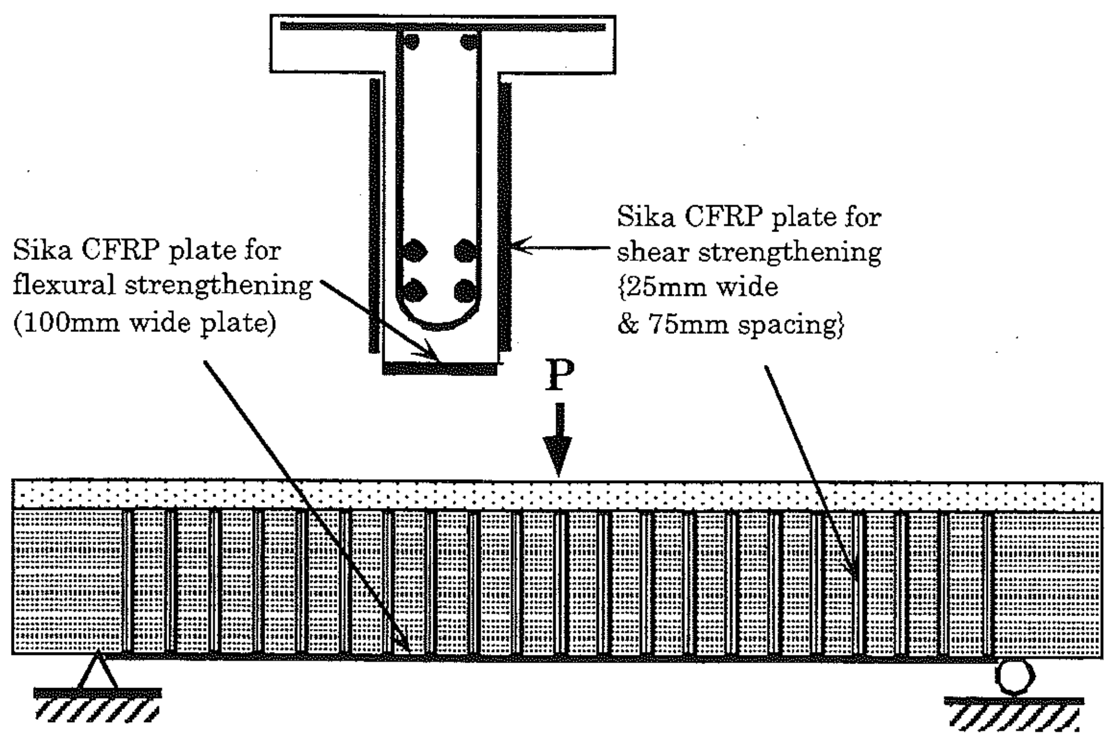


Beam No. 3

Figure 3 Test parameters for rectangular beams



Beam No. 5



Beam No. 6

Figure 4 Test parameters for T beams

For the T beams, Tonen CFRP sheet and Sika CFRP plates were used for flexural and shear strengthening. Three layers of Tonen CFRP sheet were glued on the bottom of Beam No. 5 for flexural strengthening, and one layer of CFRP sheet was glued on the web of the beam for shear strengthening. A 100 mm wide Sika CFRP plate was glued on the soffit of Beam No. 6 for flexural strengthening, and 25 mm wide Sika CFRP stripes were glued on the web of Beam No. 6 at a spacing of 75 mm. The fiber direction of all of CFRP sheets or plates was parallel to the beams for flexural strengthening and normal to the beams for shear strengthening.

For one selected set of parameters, two different shear span-to-depth ratios were used to study the influence of shear span-to-depth ratio. A shear span-to-depth ratio, 2.5, was used for rectangular beams, and 3.5 for T beams. T beams were strengthened for both flexure and shear to simulate a beam to be generally upgraded in capacity.

For all beams, the concrete surface to be glued on was ground with disk grinder for better bonding according to the recommendation of the strengthening system supplier.

2.2 Preparation of Test Beams

2.2.1 Materials

Test beams were made in the structural laboratory by the research team. Ready-mixed concrete with a target compressive strength of 34.5 MPa was requested for the test beams. However, the average compressive strength of the concrete matrix obtained from cylinder tests was about 25.4 MPa, which was less than the target strength. Table 2 summarizes the compressive strength of the concrete cylinders and the age of the beams on the test date.

Table 2 Compressive strength of concrete cylinders and time of beam test

Cylinder No.	Age, day	Compressive strength, MPa	Test beam No.	Age, day
1	26	25.3	1	24
2	26	24.7	2	24
3	26	25.0	3	24
4	26	23.8	4	25
5	26	27.0	5	25
6	26	26.4	6	25
Average	26	25.4	Average	24.5

For strengthening of the test beams, Forca Tow Sheet FTS-C1-30 (Tonen CFRP sheet) and CarboDur plate (Sika CFRP plate) with appropriate epoxy adhesives were used. The properties of the CFRP sheet or plate and adhesives are summarized in Table 3. According to the manufacturer, CFRP material is linear elastic up to failure.

Deformed steel reinforcing bars used for longitudinal reinforcement had a diameter of 10 mm (No. 10), 13 mm (No. 13), and 16 mm (No. 16) and a specific yield strength of 410 MPa with a tensile modulus of 200 GPa. No. 6 (6 mm diameter) steel reinforcing bar was used for stirrups for T beams. No. 2 reinforcing bar had a diameter of 6 mm and a specific yield strength of 280 MPa. Table 4 presents the actual yield strength and maximum tensile strength.

Table 3 Properties of CFRP sheets (plates) and adhesives.

Supplier		Tonen			Sika
System		Forca Tow Sheet			CarboDur
CFRP Sheet	Type	FTS-C1-20	FTS-C1-30	FTS-C5-30	CarboDur Strip
	Tensile Strength, N/mm	383	574	487	2868
	GPa	3.48	3.48	2.94	2.40
	Tensile Modulus, kN/mm	25.4	38.5	61.3	178.6
	GPa	228	228	372	150
	Thickness[1], mm	0.11	0.17	0.17	1.19
	Elongation, %	1.5	1.5	0.8	1.4
	Width, mm	500			50, 80, 100
	Length, m	Unlimited			Up to 250
Epoxy	Type	FR-E3P	FR-E3PS	FR-E3PW	Sikadur 30
	Application temperature, °C	Standard	Summer	Winter	≥4
	Tensile strength, MPa				24.8
	Elastic Modulus, GPa				4.48
	Elongation, %				1
	Shear strength, MPa				24.8
	Shrinkage				0.0004
	Pot-Life, min.	40	110	20	70
	Viscosity, cps	20,000	20,000	10,000	
Primer	Type	Standard, Summer, Winter, Penetrative, Summer damp surface, Winter damp surface			No Primer

1: Total cross sectional area of fibers per mm.

Table 4 Yield and maximum tensile strength of reinforcing bars.

Reinforcing bar		Yield stress, f_y MPa	Maximum stress, f_u MPa
No. 6	#6-1	315	319
	#6-2	327	330
	#6-3	335	338
	Average	325	329
No. 10	#10-1	458	652
	#10-2	454	644
	#10-3	452	656
	Average	454	650
No. 13	#13-1	513	711
	#13-2	519	721
	#13-3	530	733
	Average	521	722
No. 16	#16-1	458	741
	#16-2	455	729
	#16-3	454	733
	Average	456	734

2.2.2 Fabrication of Test Beam

Test beams were fabricated in the structural laboratory with ready-mixed concrete. All test beams had four longitudinal reinforcing bars, placed in two rows; two in the lower row at a clear distance of about 51 mm from the bottom fiber and two in the upper row with a clear distance of 40 mm from the lower row. Two strain gages were attached on the lower two reinforcing bars at the midspan location of each beam before assembling the reinforcement cage. For T beams, two-leg closed stirrups made of No. 6 reinforcing bar were placed at a spacing of 75 mm throughout the beams. Figure 5 shows steel cages and wood molds.

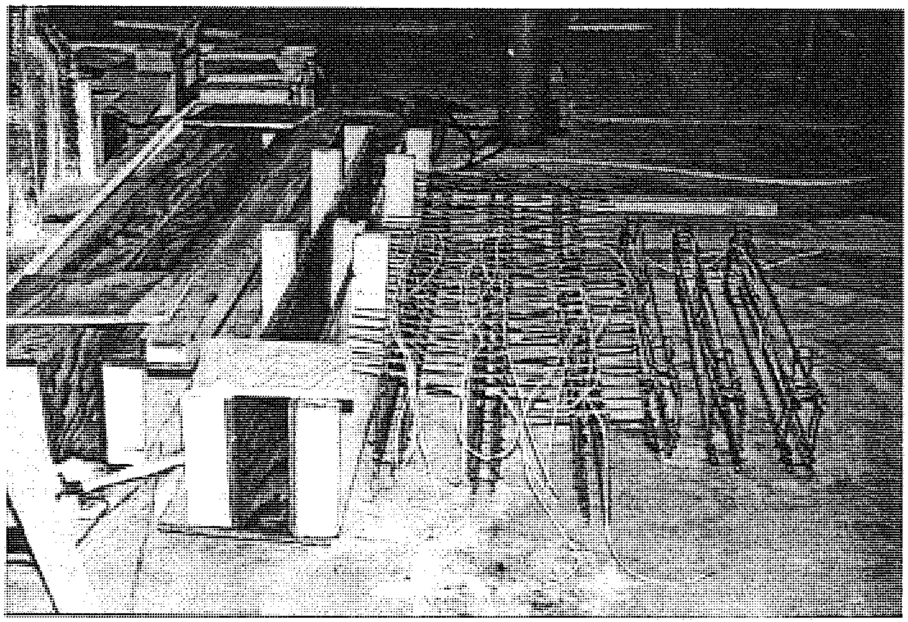


Figure 5 Steel reinforcement cages and wood molds for test beams

One batch of ready-mixed concrete was used for all test beams. After casting of concrete, the beams were covered with plastic sheet to keep moisture inside the beams. The test beams were removed from the molds 3 days after casting of concrete and stored in the laboratory.

The concrete surface to be bonded to was ground enough to remove laitance and provide open texture of aggregates by disk grinding. After grinding, dust and cement particles were removed by brushing and vacuum cleaning. Tonen CFRP sheet and Sika CFRP plate was cut to proper length by sharp knife and disk cutter, respectively. The CFRP sheet and plate was cleaned with a white cloth before gluing to remove soiling as well as carbon dust.

Adhesives were mixed according to the technical data sheets provided by the strengthening system supplier. For Tonen strengthening system, primer and epoxy adhesive were applied using a roller according to the commercial specifications. A hard roller was used to press the CFRP sheet into the epoxy adhesive. For the Sika strengthening system, a trowel was used to apply the epoxy adhesive to the surface of beam soffit or web and CFRP plate. Also, a hard roller was used to press the CFRP plate into the adhesive and to force any air pockets out from the interface. Note that the beams were not precracked before applying the CFRP plates or sheets. Figure 6 shows the test beams after gluing CFRP sheet or plate.

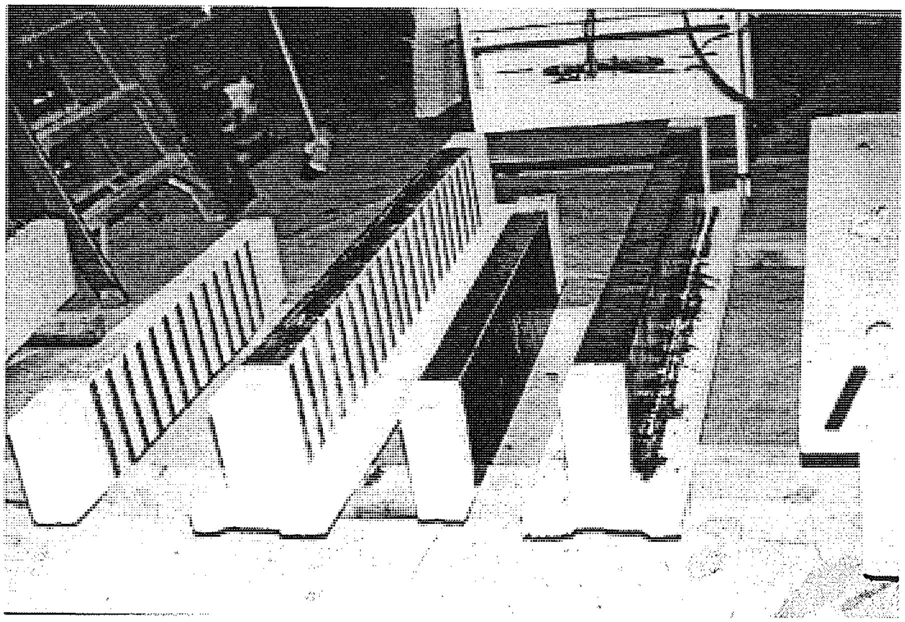


Figure 6 Test beams after gluing CFRP sheet or plate

2.3 Data Acquisition and Test Procedure

Figure 7 shows instrumentation layout for shear test. A computer data acquisition system (Megadack System) was used to measure the load and corresponding deflection as well as strains of longitudinal reinforcing bar, stirrup, and CFRP sheet or plate for flexural and shear strengthening.

Each test beam was loaded monotonically up to failure using displacement control at a loading rate of 0.025 mm per second by the Instron loading machine having a capacity of 450 kN. Each beam was pre-loaded to about 8.9 kN before testing to remove any residual stress and deformation in the test beam and stabilize the instrumentation. At every 8.9 kN interval, loading was temporarily stopped to observe the development of cracks and to mark them. All test beams were loaded monotonically up to their failure.

The following data was obtained every second by the data acquisition system:

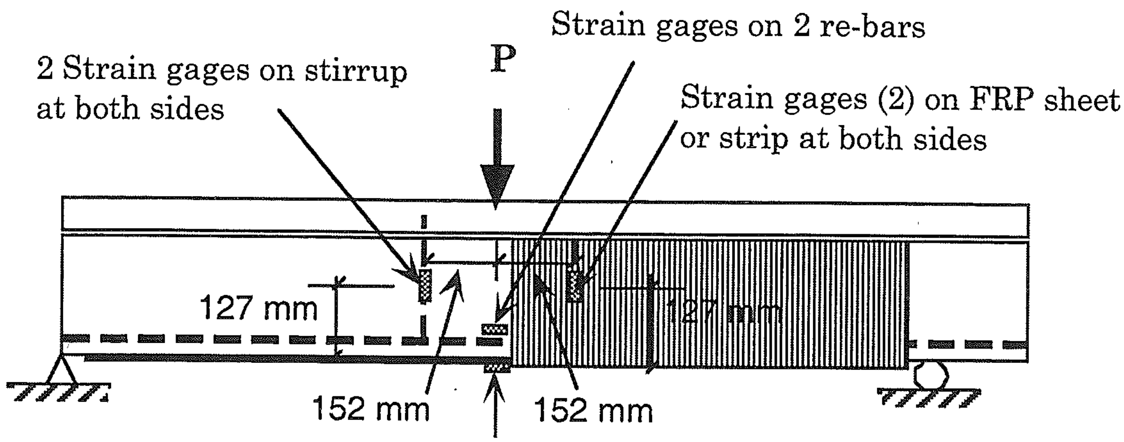
(1) load and deflection from the Instron loading machine

(2) strains of longitudinal reinforcing bars at midspan

(3) strains of both legs of single stirrups

(4) strains of CFRP sheet or strip on the web of beams (2 strains)

(5) strains of CFRP sheet or plate on the bottom of beams at the middle of span



Note: Location of strain gages for rectagular beams is the same as T beams

Figure 7 Instrumentation layout for shear test

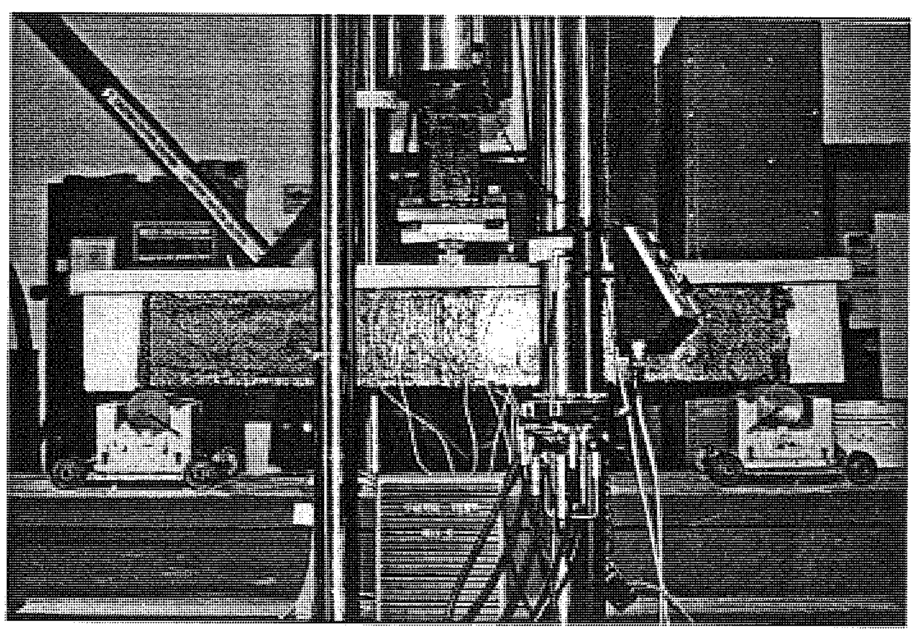


Figure. 8 Shear test set-up for T beam strengthened with CFRP sheet

3 ANALYSIS AND DISCUSSION OF TEST RESULTS

Relevant parameters and test results are summarized in Tables 5a and 5b.

3.1 Rectangular Beams

To evaluate the effect of shear strengthening using CFRP sheet or strips in rectangular beams, the test results of Beams No. 2 and No. 3 are compared with those of the control beam, Beam No. 1. Beams No. 2 and No. 3 were made without shear reinforcement (no steel stirrups) and were strengthened using one layer of Tonen CFRP sheet and or 25 mm wide Sika CFRP plate at a spacing of 75 mm, respectively. To ensure shear failure and avoid flexural failure prior to shear failure, three rectangular beams were initially reinforced with the maximum flexural reinforcement ratio according to the AASHTO code, assuming a target compressive strength of 35 MPa. After recalculations using the actual concrete and steel strength, the beam had a 141% reinforcement ratio (ρ_{max}) compared with the maximum value given by AASHTO. All beams were subjected to a concentrated load at midspan with a shear span-to-depth ratio of 2.5.

Control beam, Beam No. 1 failed by shear (diagonal tension failure) long before yielding of the longitudinal reinforcing bars. A critical diagonal tension crack occurred at a load of about 52 kN; which suddenly developed from the loading point at midspan to the support, resulting in diagonal tension failure (Figure 9).

Table 5a Summary of test results of shear test

Beam No.	Section type	Shear reinforcement (A_v)		a/d	Longitudinal Reinforcement	Failure Mode	Ultimate load, kN	Ultimate deflection, mm	Predicted failure load (bending failure)[1] kN	Predicted failure load (shear failure)[2] kN
		Stirrup	CFRP							
1	Rectangular	None	None	2.5	1.41 ρ_{max}	Shear failure	51.8	3.0	106.75	31.14
2	Rectangular	None	CFRP sheet (Tonen)	2.5	1.41 ρ_{max}	Steel yielding & Delamination	130.5	7.4	106.75	615.16
3	Rectangular	None	CFRP strip (Sika)	2.5	1.41 ρ_{max}	Delamination shear failure	88.1	4.3	106.75	1001
4	T-section	0.91 A_v max (64.5 mm^2)	None	3.5	0.89 ρ_{max}	Steel yielding & shear failure	164.2	15.5	161.75	166.71
5	T-section	0.91 A_v max (64.5 mm^2)	CFRP sheet (Tonen)	3.5	0.89 ρ_{max}	Steel yielding & Delamination	240.2	12.2	257.15	745.75
6	T-section	0.91 A_v max (64.5 mm^2)	CFRP strip (Sika)	3.5	0.89 ρ_{max}	Delamination shear failure	214.5	9.9	301.35	1131.75

Note: 1. See Appendix for detail calculations

2. See Appendix for detail calculations

Table 5b. comparison actual to controlling predicting failure loads

Beam #	1	2	3	4	5	6
Predicted Failure load (kN)	31.14	106.75	106.75	161.75	257.15	301.35
Actual Failure load (kN)	51.8	130.5	88.1	164.2	240.2	214.5
Ratio actual/predicted	1.66	1.22	0.83	1.02	0.93	0.71

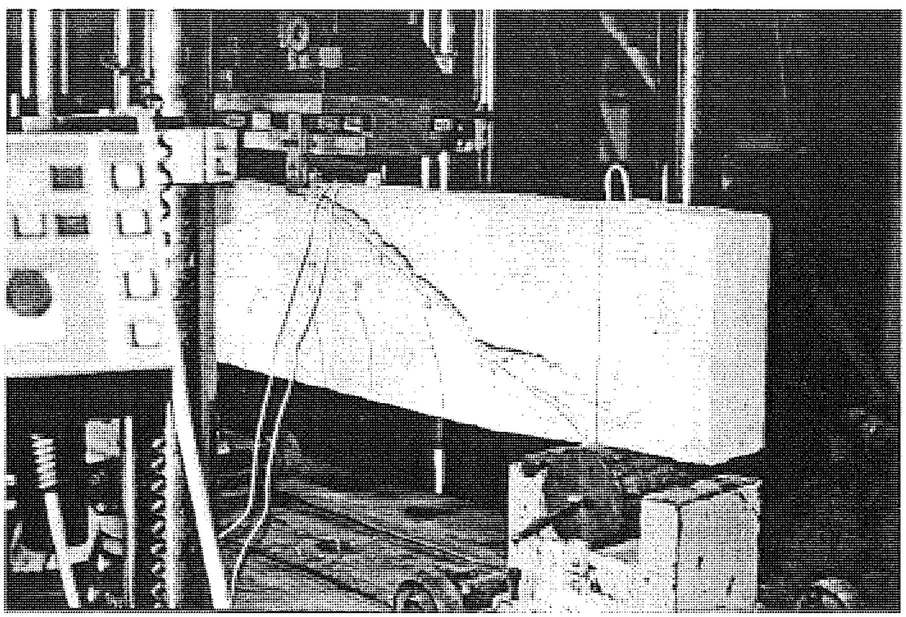


Figure 9 Diagonal tension failure of Beam No. 1

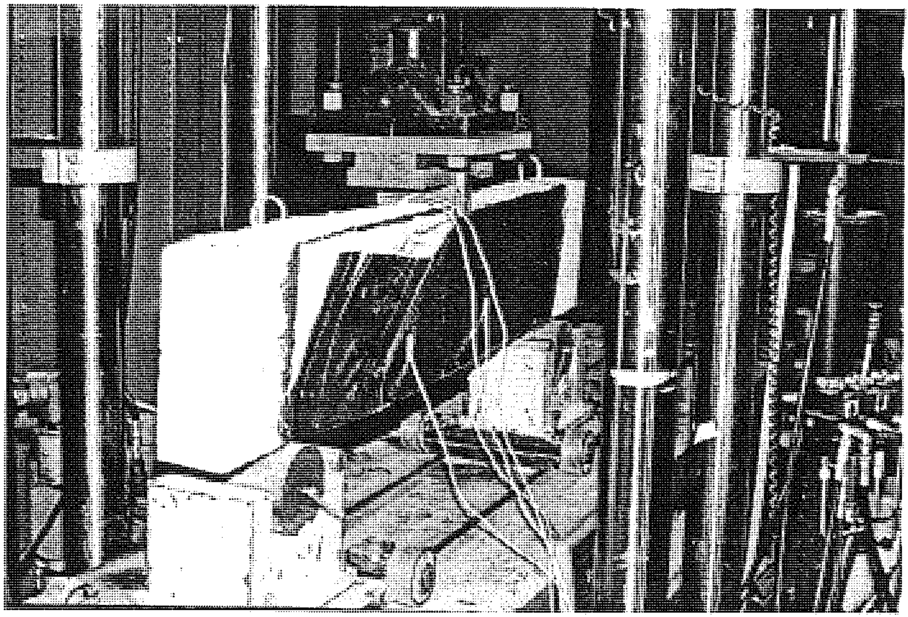


Figure 10 Shear-failure after delamination of CFRP sheet in Beam No. 2

Beam No. 2 with Tonen CFRP sheet, failed by shear-compression failure after delamination of the CFRP sheet used for shear strengthening. Shear-compression failure is characterized by crushing of the concrete in the compression zone at the end of the shear crack. Delamination occurred from the top of the web at maximum load, about 130 kN, after yielding of the longitudinal reinforcing bars. One side of the CFRP sheet was completely separated from the web due to the spalled concrete at the compression failure. Beam No. 2 attained its nominal flexural load carrying capacity, because reinforcement yielding occurred before the delamination of CFRP sheet. Thus, strengthening for shear with CFRP sheet significantly increased the ultimate load and deflection in beams with no existing shear reinforcement as shown in table 5b. Figure 10 shows the shear-compression failure of Beam No. 2 after delamination of the CFRP sheet.

Beam No. 3 with the Sika CFRP strips bonded to the web failed by diagonal tension failure just after delamination of some of the CFRP strips. Delamination suddenly occurred at a load of about 88 kN, before yielding of the longitudinal reinforcement. The shorter length side (with respect to the diagonal crack) of the CFRP strip at the critical diagonal crack was delaminated. The critical diagonal crack had a angle of about 40 degrees. Figure 11 shows the diagonal tension failure due to delamination of CFRP strip.

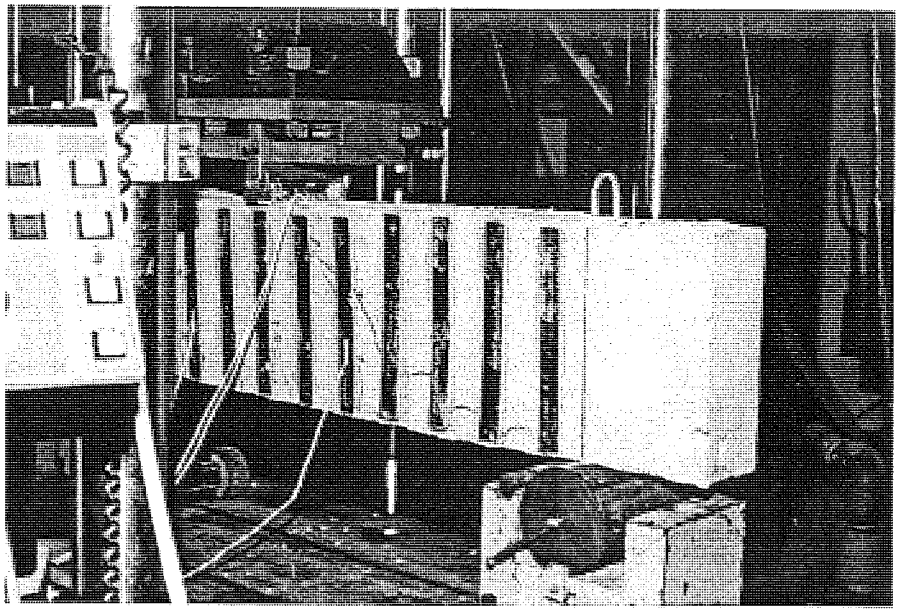


Figure 11 Shear failure due to delamination of CFRP strip in Beam No. 3

Figure 12 gives the load-deflection curves of the control beam and two shear strengthened beams using the two different strengthening systems. Beam No. 1 shows linear elastic behavior up to diagonal tension failure. Due to this premature failure by shear, Beam No. 1 had the lowest load carrying capacity of the three beams.

Beam No. 2 strengthened for shear with the Tonen CFRP sheet had the highest load carrying capacity and ductility of the three beams. The CFRP sheet used for shear strengthening significantly increased the maximum ultimate load and helps to maintain this maximum load up to relatively high deflection values before failure. The ultimate load of Beam No. 2 was about 250% that of the control beam and its deflection at ultimate was about 240% that of the control beam (Table 5a and Figure 12). The reason for the high load carrying capacity and ductility is that the CFRP sheet, by providing shear strengthening, prevented the beam from failing by shear prior to yielding of the longitudinal reinforcement. The yielding of reinforcing bars before shear failure is confirmed for beam No. 2 (2 gages) in Figures 13 and 14 that show the load-strain and deflection-strain curves of the reinforcing bars (strains at failure were larger than 0.2%, yielding strain for steel).

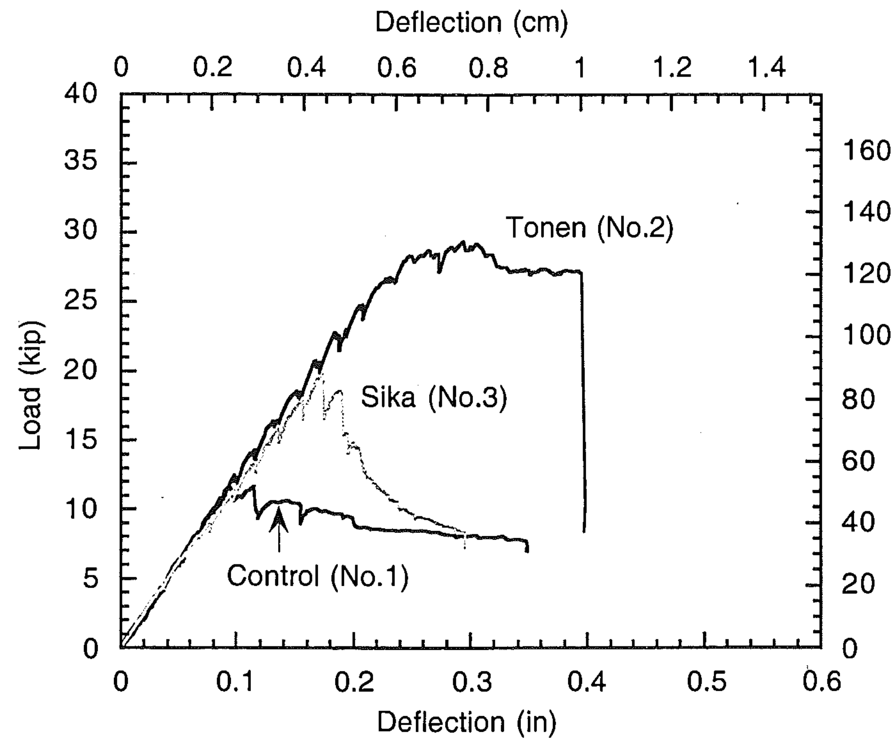


Figure 12 Load-deflection curves of rectangular beams

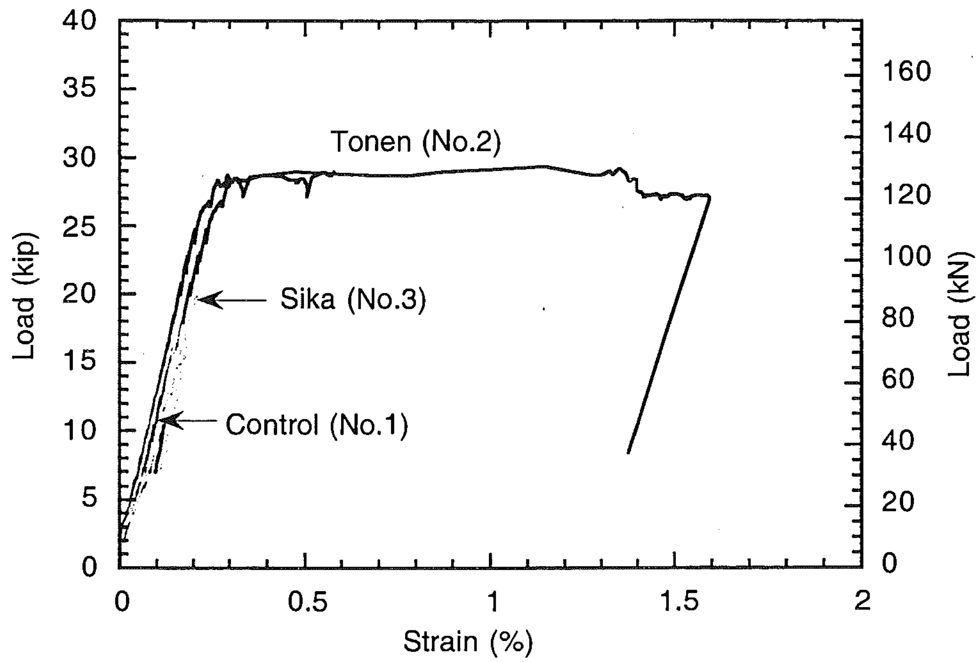


Figure 13 Load-strain curves of reinforcing bar in rectangular beams

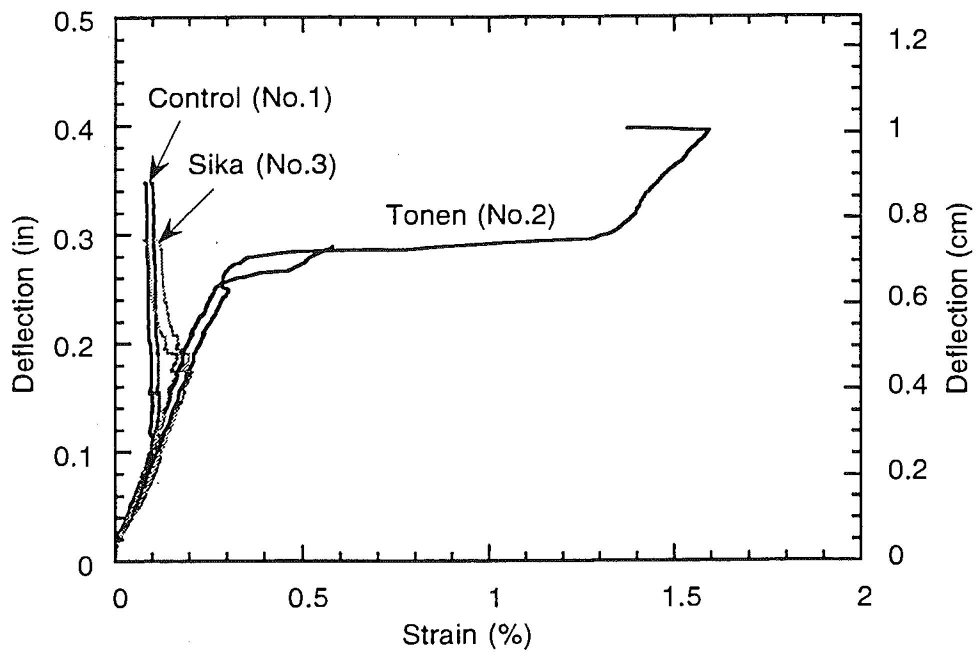


Figure 14 Deflection-strain curves of reinforcing bar in rectangular beams

Beam No. 3 with the Sika CFRP strip had higher load carrying capacity than the control beam, Beam No. 1, but had a lower load capacity than Beam No. 2. The beam had about 70% higher ultimate load and 40% higher ultimate deflection than the control beam (Table 5a and Figure 12). This is because shear failure was delayed by the CFRP strip used for shear strengthening, but it occurred before yielding of the flexural reinforcement.

Delamination of the Sika CFRP strip at a lower load than for Beam No. 2 is attributed to the fact that the strips were not anchored by wrapping around the web as with the Tonen sheets, and their bonded area was smaller in spite of equivalent shear strengthening. It is likely that L shaped strips will improve their anchorage and lead to a more effective strengthening. Figure 13 also shows that the longitudinal reinforcing bars in Beam No. 3 were within the elastic range at failure (strain less than yield) .

Figures 15 and 16 show the load-strain and deflection-strain curves of CFRP sheet or plate (strip) for shear strengthening. It can be observed that the CFRP sheet or strip contributed to shear resistance after shear cracking. The steep initial portion of the curves likely represent the load prior to cracking while the second portion corresponds to loads where microcracks have occur. Tonen CFRP sheet and Sika CFRP strip had an average strain of about 0.22 % and 0.08 % at the maximum ultimate load, respectively. These strains are much smaller than the tensile strain capacity at failure, 1.5% and 1.4 % respectively, of the Tonen CFRP sheet and Sika CFRP plate.

The strain of 0.22% in the Tonen CFRP sheet corresponds to 500 MPa tensile stress, while the strain of 0.08% in the Sika CFRP plate corresponds to 120 MPa tensile stress. Considering the area of CFRP sheet and strip, tensile forces in the Tonen CFRP sheet and Sika CFRP strip are 82.3 N/mm and 143.6 N/mm, respectively. It is believed that a 25 mm Sika CFRP strip involves a higher resisting width of concrete and thus seems more efficient. This could be confirmed from observing the Sika strip that separated from the web, after testing. Indeed a 25 mm Sika CFRP strip tore out a strip of concrete cover wider than 25 mm.

Dividing the tensile force per unit width by the bonded length of CFRP sheet or strip gives the shear resisting stress of concrete, based on the assumption of uniform shear stress distribution. Therefore, shear stresses of concrete at delamination were 0.32 MPa and 0.27 MPa for Tonen CFRP sheet and Sika CFRP strip, respectively. These values are of the order of $0.063\sqrt{f_c'}$ and $0.054\sqrt{f_c'}$, respectively, assuming $f_c' = 25.40MPa$. Given the limited number of tests with the Sika system, this procedure seems reasonable at this time.

Figures 15 and 16 show that, in beams without existing steel stirrups, the strains in the CFRP sheet and strip vary roughly linearly with the load and deflection prior to the onset of delamination.

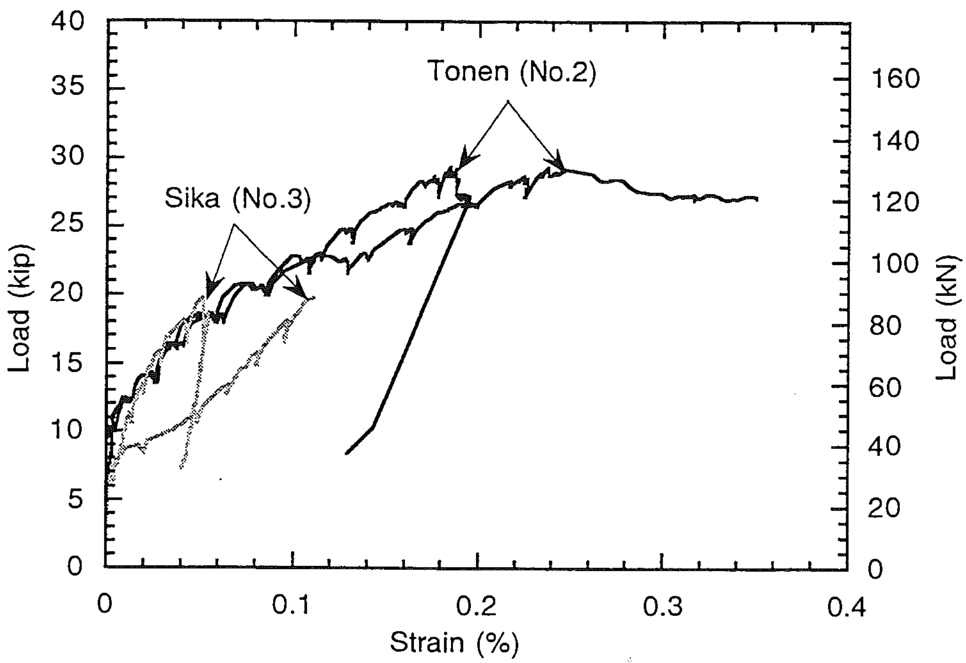


Figure 15 Load-strain curves of CFRP sheet (strip) for shear in rectangular beams

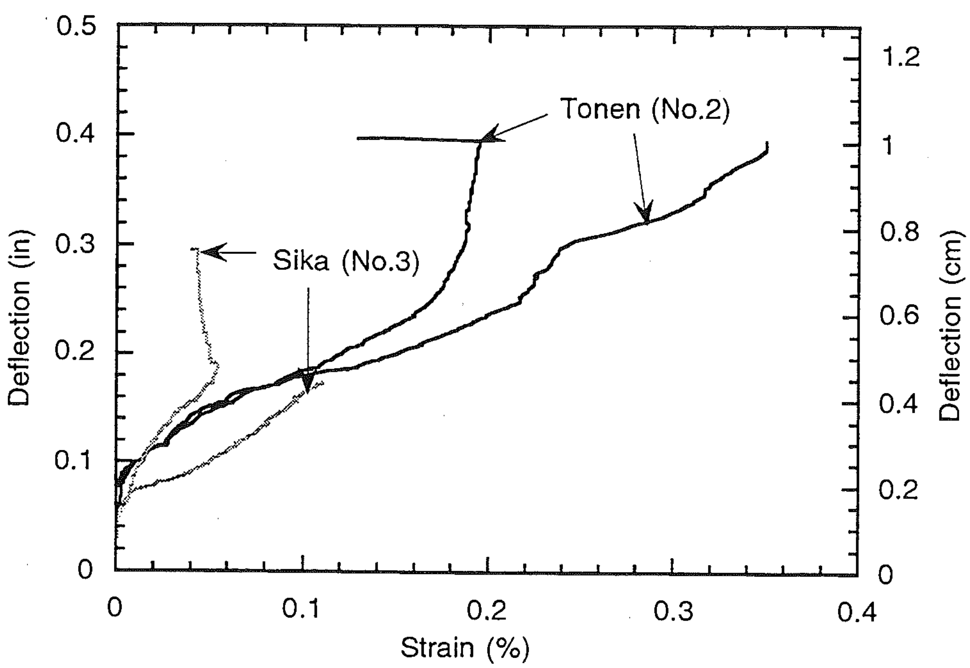


Figure 16 Deflection-strain curves of CFRP sheet (strip) for shear in rectangular beams

3.2 T-section Beams

To evaluate the effect of combined flexural and shear strengthening using CFRP sheet or plate, the test results of Beams No. 5 and No. 6 are compared with those of the control beam, Beam No. 4. Steel stirrups were provided for all three T beams in amounts corresponding to 91% of the required shear reinforcement for a reinforced concrete beam to resist a load at midspan that is allowed with longitudinal reinforcement equal to the maximum allowable by the AASHTO code (ρ_{max}). Beams No. 5 and No. 6 were strengthened using one layer of CFRP sheet (Tonen CFRP sheet) and 25 mm wide CFRP strip (Sika CFRP Plate) at a spacing of 75 mm, respectively. Also, all three T beams were intended to be provided with about one half of maximum flexural reinforcement according to the AASHTO code. Recalculations using the actual concrete and steel strength led to a new value of 89% the maximum flexural reinforcement. All beams were subjected to a concentrated load at midspan with a shear span-to-depth ratio of 3.5. In their failure modes, the control beam, Beam No. 4 failed by compression strut failure after yielding of the longitudinal reinforcing bars. At a load of about 133 kN to 156 kN, several diagonal cracks developed from one of the supports resulting in crushing failure of compression strut near the support. The compression strut had an angle of about 40 degrees. Figure 17 shows the compression strut failure of Beam No. 4.

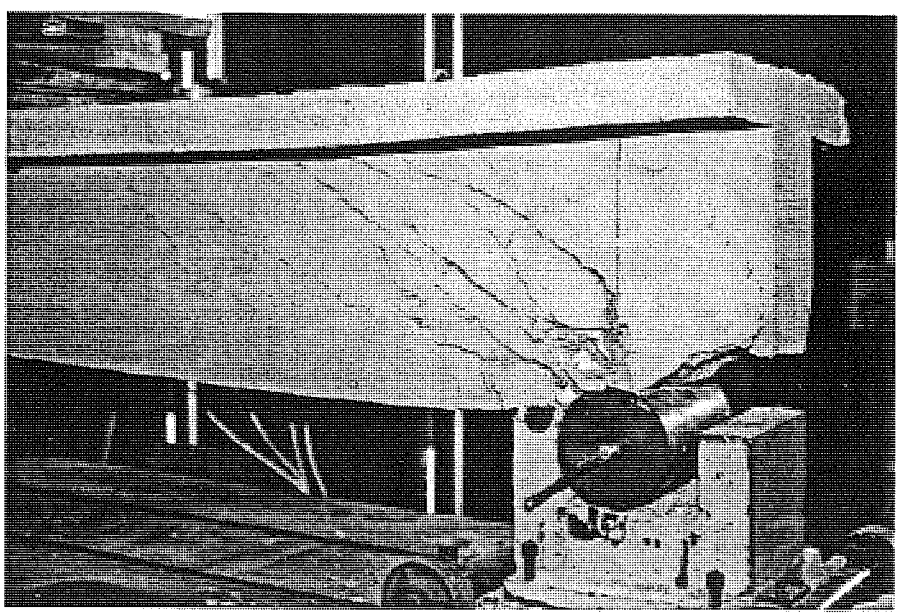


Figure 17 Compression strut failure of Beam No. 4

Beam No. 5, reinforced with Tonen CFRP sheet, failed by shear-compression failure after delamination of the CFRP sheet (that was bonded to the web for shear strengthening) and subsequent tensile failure of CFRP sheet that was bonded to the bottom for flexural strengthening. Delamination of the CFRP sheet bonded to the web started at a load of about 240 kN, just after yielding of longitudinal reinforcing bars. One side of the CFRP sheet was completely separated from the web due to the compression failure of concrete. Beam No. 5 exhibited the highest ultimate load of the three T beams.

Note that the CFRP sheet for flexural strengthening ruptured after delamination of the CFRP sheet for shear strengthening. This tensile rupture is attributed to stress concentration due to dowel action at a critical inclined crack plane. Indeed the measured strain at tensile failure of the CFRP sheet was about 0.9 % at midspan, which was smaller than the 1.5 % tensile failure strain. The critical inclined crack developed with an angle of about 50 degrees from the loading point at midspan. The beam finally failed by shear-compression after tensile failure of the CFRP sheet used as flexural reinforcement. Figure 18 shows the shear-compression failure of the beam.

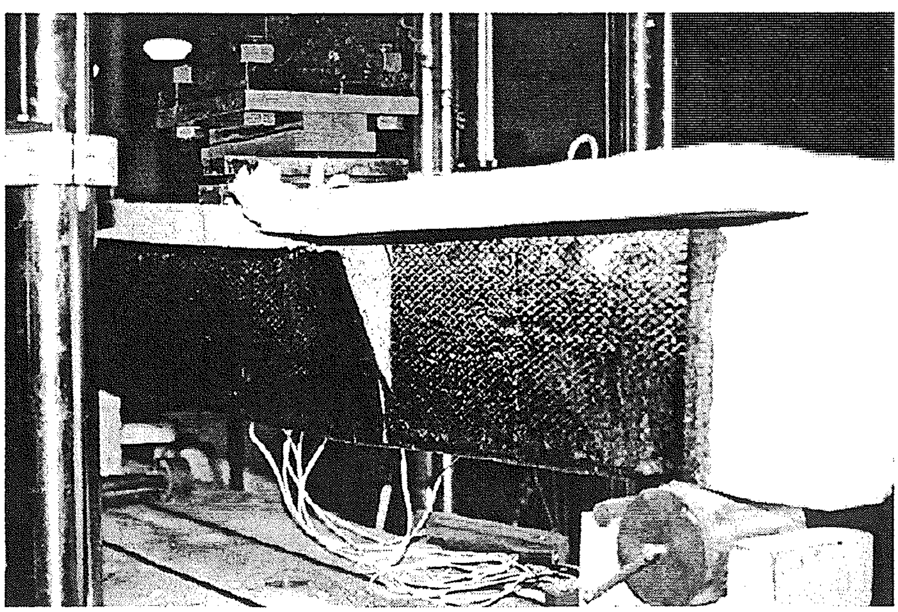


Figure 18 Shear-failure after delamination of CFRP sheet in Beam No. 5

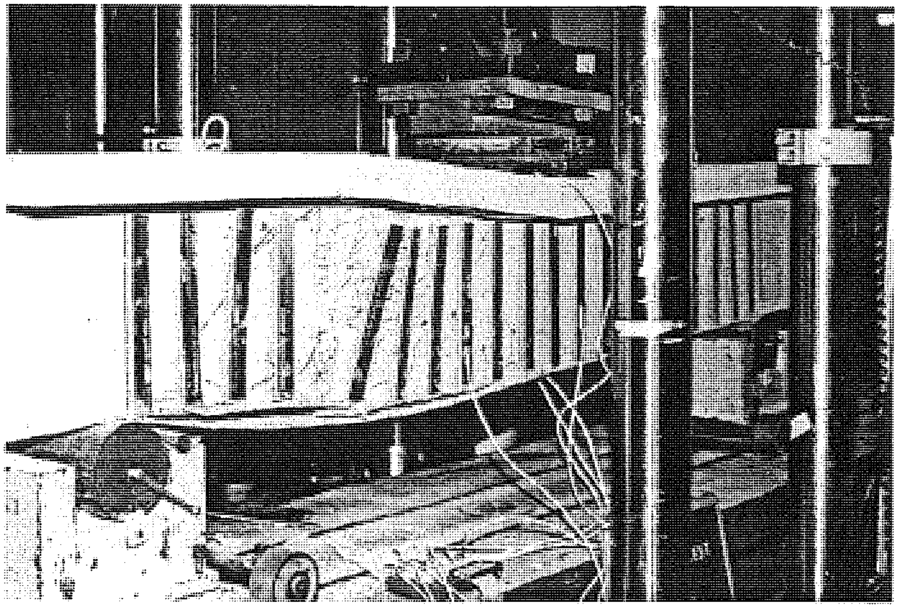


Figure 19 Shear failure due to delamination of CFRP strip in Beam No. 6

Beam No. 6 with Sika the CFRP plate failed by compression strut failure after delamination of the CFRP plate bonded to the web and the CFRP plate bonded to the bottom of the beam. Delamination occurred progressively starting at loads about 133 kN up to the ultimate load, about 214 kN, and before yielding of longitudinal reinforcement. At a load of 214 kN, the CFRP plate for flexural reinforcement suddenly delaminated. Beam No. 6 had higher ultimate load than Beam No. 4 (control beam), but a lower ultimate load than Beam No. 5. The delamination of CFRP strips for shear strengthening resulted in large shear deformation in the delaminated area of the web and led to the delamination of the CFRP plate for flexural strengthening. Figure 19 shows the compression strut failure in Beam No. 6.

Figure 20 shows the load deflection curves of the control beam and the two flexure and shear strengthened beams with the Tonen and Sika system. Beam No. 5 which was strengthened for both flexure and shear with Tonen CFRP sheet had the highest load carrying capacity of the three beams. It had an ultimate load about 45% higher than that of the control beam. The reason for the high load carrying capacity is that the CFRP sheet for flexural strengthening increased flexural resistance and the CFRP sheet for shear strengthening delayed shear failure resulting in a higher ultimate load. Yielding of the longitudinal reinforcing bar occurred before shear failure as shown in Figure 21.

Beam No. 6 with Sika CFRP plate had a higher load carrying capacity than the control beam, Beam No. 4, but had a lower load carrying capacity than Beam No. 5. Its ultimate load was about 30% higher than that of the control beam but 10% less load than that of Beam No. 5. For the same reason explained earlier for Beam No. 3, delamination of CFRP shear strip at a lower load than Beam No. 5 is attributed to lack of anchorage and smaller bonded area. As shown in Figure 20, Beam No. 6 abruptly lost its load carrying capacity due to delamination of the CFRP plate for flexural strengthening. At the time of failure or sudden drop in load, the longitudinal reinforcing bars were within their elastic range as shown in Figure 21.

Figures 23 and 24 show that the strain in the stirrups were in the linear elastic range from the onset of shear cracking to just before the onset of delamination of the CFRP sheet or strip used for shear strengthening. Indeed the maximum strain was in all cases less than 0.15% that is less than ε_y=0.2% (yielding strain).

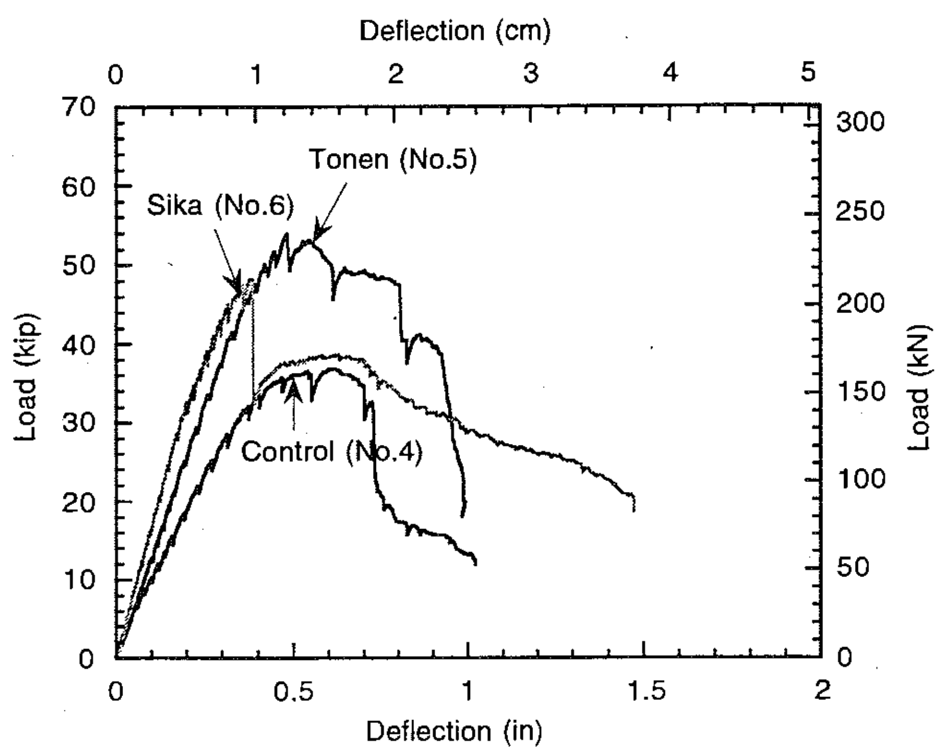


Figure 20 Load-deflection curves of T beams

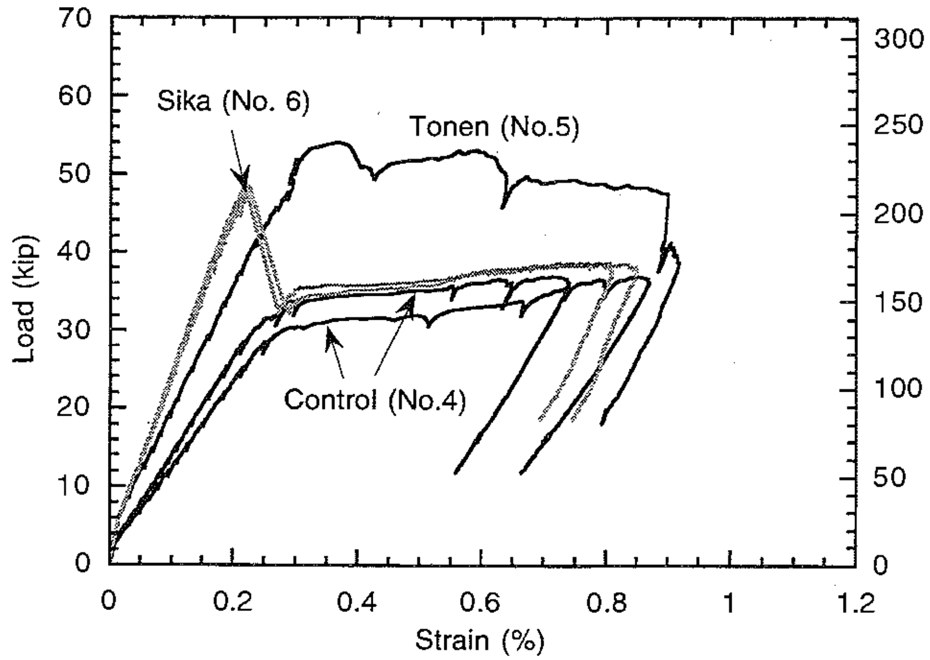


Figure 21 Load-strain curves of reinforcing bar in T beams

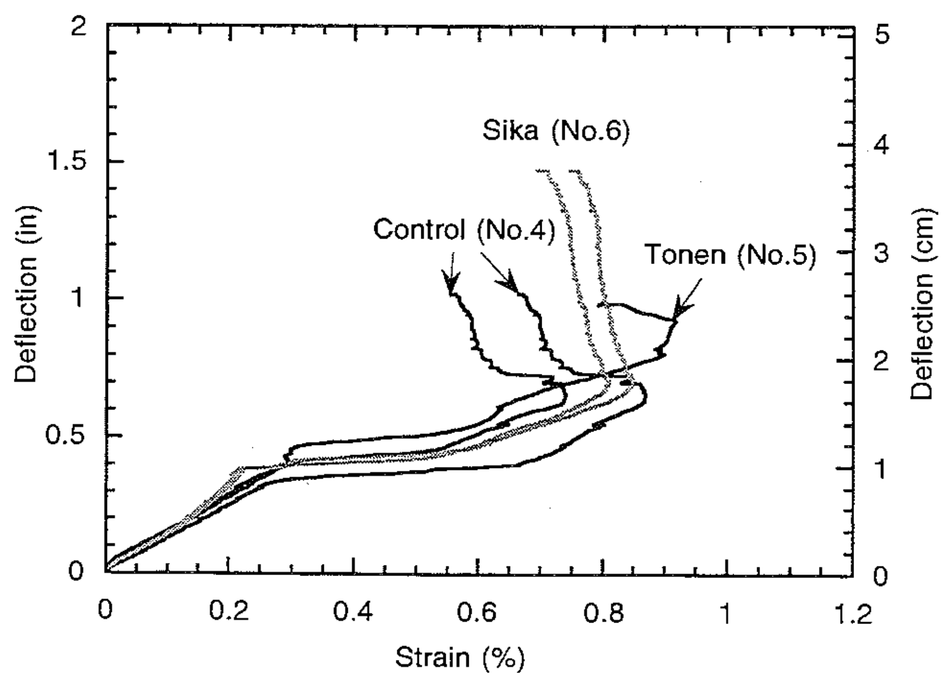


Figure 22 Deflection-strain curves of reinforcing bar in T beams

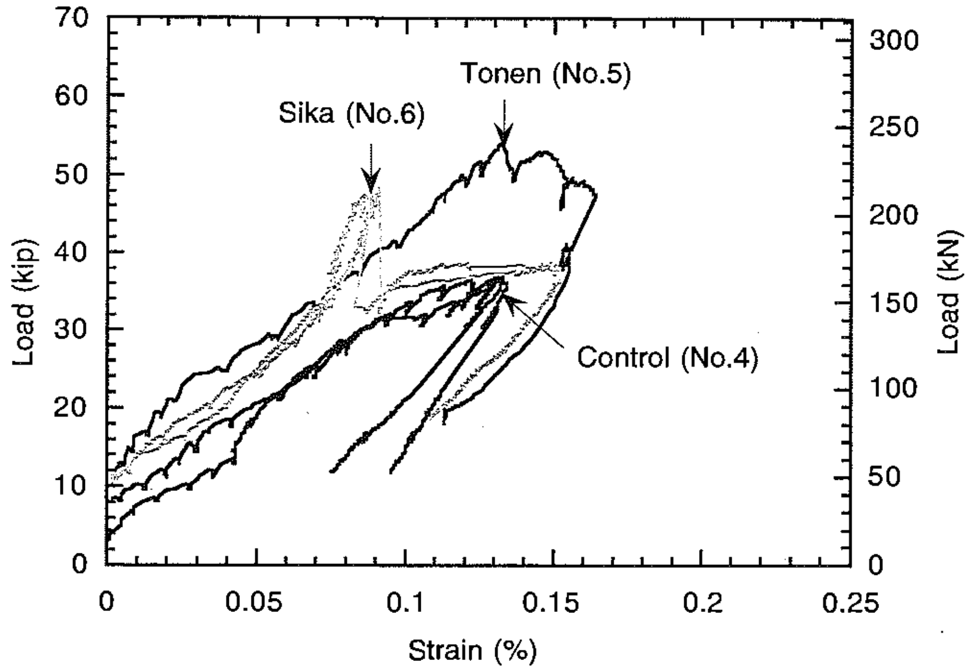


Figure 23 Load-strain curves of stirrups in T beams

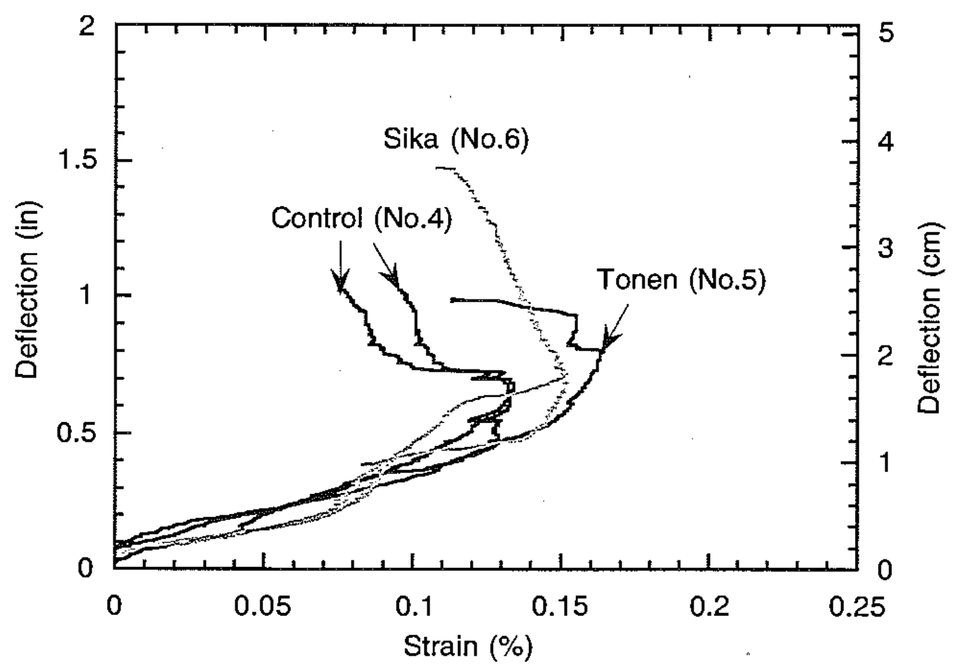


Figure 24 Deflection-strain curves of stirrups in T beams

Figures 25 and 26 show the load-strain and deflection-strain curves in CFRP sheet or plate used for shear. It can be observed that the CFRP sheet or strip contributed to shear resistance immediately after shear cracking and the contribution (strain) was almost proportional to the load up to the onset delamination. Tonen CFRP sheet and Sika CFRP strip had an average strain at the maximum load of about 0.21% and 0.07 %, respectively.

The strain of 0.21% in Tonen CFRP sheet corresponds to 478 MPa tensile stress, while the strain of 0.07% in Sika CFRP plate corresponds to 105 MPa tensile stress. Considering the area of CFRP sheet and strip, tensile forces in Tonen CFRP sheet and Sika CFRP strip are 78.8 N/mm and 126.1N/mm, respectively. Dividing the tensile force per unit width by the length of CFRP sheet or strip gives the average shear resisting stress of the concrete based on the assumption of uniform shear stress distribution. Average shear stresses of concrete observed at onset of delamination are 0.31 MPa and 0.50 MPa for Tonen CFRP sheet and Sika CFRP strip, respectively. These values are of the order of $0.062\sqrt{f_c^{'}}$ and $0.10\sqrt{f_c^{'}}$, respectively ($f_c^{'} = 25.4MPa$).

As shown in Figures. 27 and 28, the tensile strains of CFRP sheet and plate used for flexure were also proportional to the load and deflection before the onset of delamination of the CFRP sheet and strip used for shear strengthening.

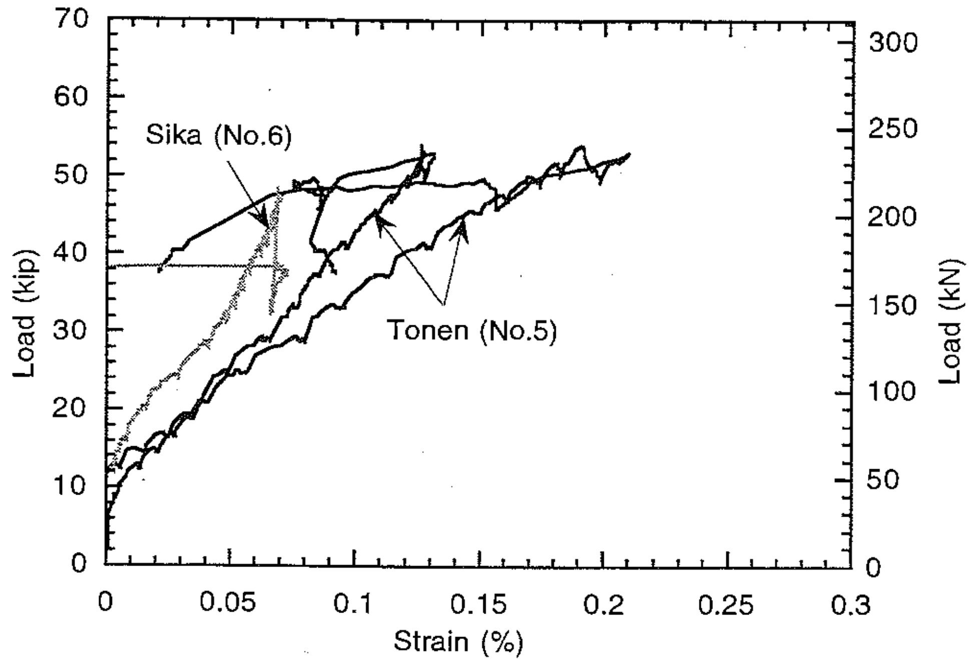


Figure 25 Load-strain curves of CFRP sheet or strip for shear in T beams

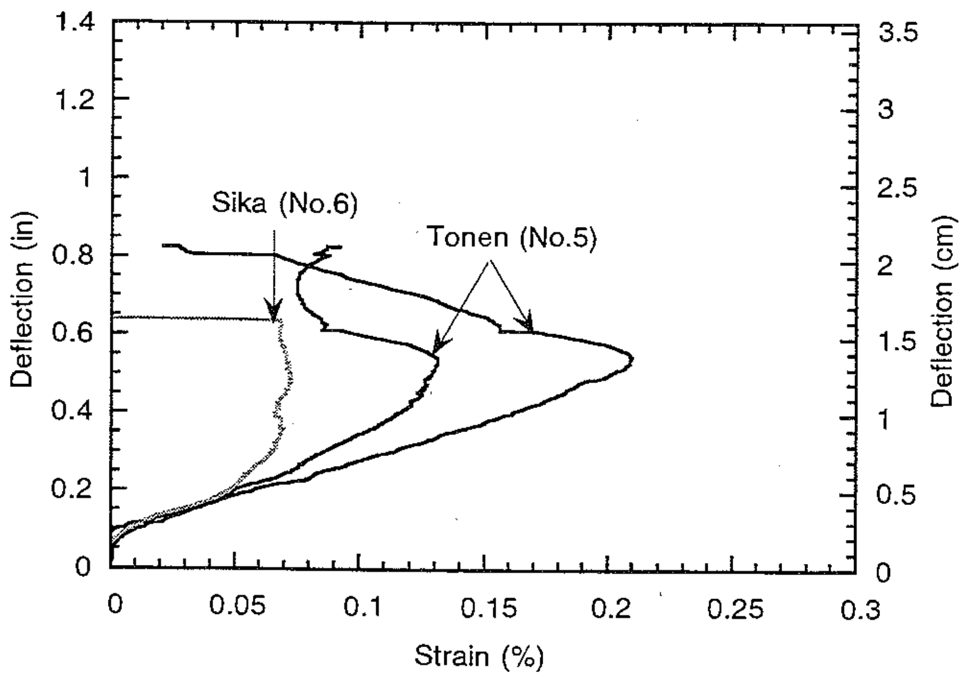


Figure 26 Deflection-strain curves of CFRP sheet or strip for shear in T beams

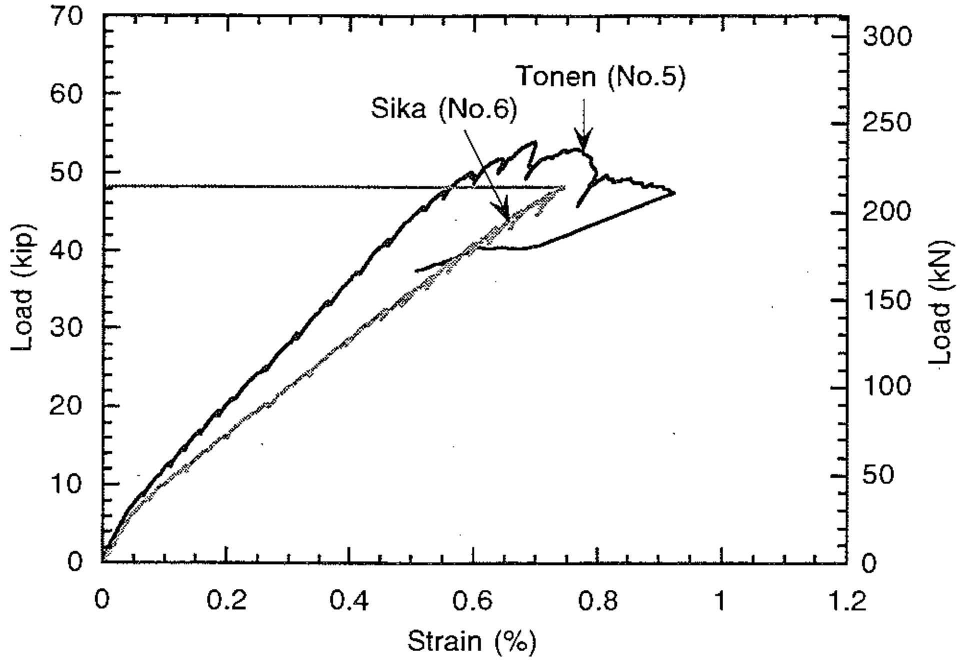


Figure 27 Load-strain curves of CFRP sheet or plate for flexure in T beams

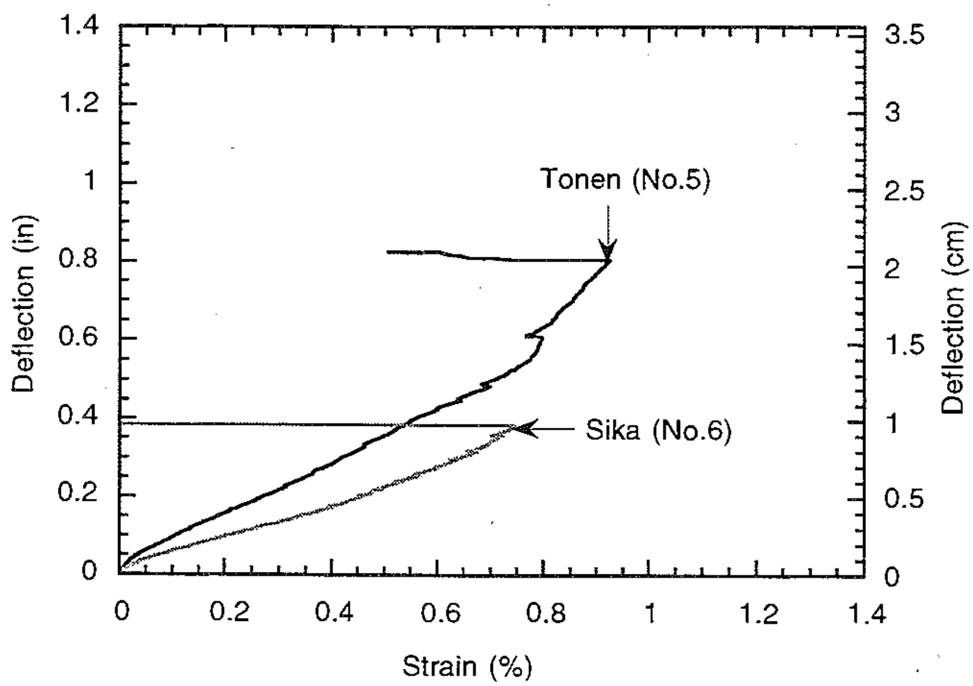


Figure 28 Deflection-strain curves of CFRP sheet or plate for flexure in T beams

4. CONCLUSIONS

This investigation dealt with the shear behavior of reinforced concrete beams strengthened using bonded carbon fiber reinforced plastic (CFRP) sheets or plates. Based on the observation and analysis of the experimental test results the following conclusions are drawn.

1) The strengthening for shear using externally bonded CFRP sheets or plates can significantly improve the ultimate loading capacity of reinforced concrete beams having deficiency in shear. Because shear failure is delayed, their ultimate deflection is also significantly increased. In beams insufficiently reinforced for shear, the use of CFRP shear strengthening led to an increase in load capacity of at least 30%.
2) The Tonen CFRP sheet led to a higher shear strengthening effect than the Sika CFRP because of its larger bond area and because it was better anchored by wrapping around the web. The development of L or Z shaped Sika CFRP plates should improve the strip anchorage and contribute to its increased efficiency.
3) It was generally observed that shear strengthened beams fail by delamination of the CFRP sheet or plate used for shear strengthening, resulting in shear failure of concrete.
4) The tensile stresses generated in the bonded CFRP sheet or plate used for shear are very low compared to their tensile strengths. In this study they were about one twentieth and one seventh the tensile strength of CFRP Tonen sheet and Sika plate, respectively.
5) In this study, shear stresses of concrete at onset of delamination of CFRP sheet and plate used for shear were of the order of about $0.06\sqrt{f_c'}$ and $0.10\sqrt{f_c'}$ for Tonen CFRP sheet and Sika CFRP plate, respectively.

6) The strains in the steel stirrups and the CFRP sheets or plates used for shear varied linearly with the applied load in the range following shear cracking and before onset of delamination.

7) The stresses in the CFRP sheet or plate used for flexure increased linearly with the load and deflection in the range prior to delamination of CFRP sheet and plates used for shear.

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6. APPENDIX A

Moment Capacity Calculations

- **Input Information**

- Distance from centroid of steel to top layer of concrete

For beam No.1-3:

d_1=(10-2-4/8*1/2)*25.4 = 197 mm, d_2=(10-2-4/8*1/2-1.5)*25.4 = 159 mm

d_e = (7.75*0.4+6.25*.22)/0.62*25.4 = 183 mm

For beam No.4-6:

d_1=(12-2-5/8*1/2)*25.4 = 246 mm, d_2=(9.6875-1.5)*25.4 = 208 mm

d_e = (9.6875*.62+8.1875*0.4)/1.02*25.4 = 231 mm

- Concrete compressive strength = 25.4 MPa , β_1 = 0.85
- Steel yield strength:

For beams 1-3, f_y = (65.9*0.22+75.5*0.4)/0.62*6.895 = 496 MPa

For beams 4-6, f_y = (75.5*0.4+66.1*0.62)/1.02*6.895 = 482 MPa

- **Computation of As Balanced**

Rectangular beams $\rho_b = 0.85*\beta_1*f'_c/f_y*(e_{cu}/e_{cu}+e_y) = 0.02021$

$\rho_{max} = 0.75\rho_b = 0.01515$, $A_{smax} = 283\ mm^2$

A_s (used) = 400 mm^2

T-section behavior:

A_s balanced = Concrete force due to equilibrium(C_c) /f_y

For d_e = 231 mm, c_b **(neutral axis)** for balanced condition = 137 mm> flange height (hf = 51 mm). Find C_c for T-section:

C_c = (b-bw)*hf*0.85*f'c + 0.85*f'c*bw*β_1*c_b

C_c = 477 kN

A_s balanced = 477,000/482 = 987 mm^2 , A_{smax} = 0.75 A_s balanced = 742 mm^2

A_s (used) = 658 mm^2

- **Computation of M_{max}**

M_{max} = A_{smax}*f_y*(d_e-a/2), where a=A_{smax}*f_y/(0.85*f_y*bf)

if a>hf T-section behavior, therefore this equation can not be applied.

-<u>T-section beams</u>:

For A_{smax} = 742 mm^2, d_e = 231 mm, a = 61 mm > 51 mm, T-section,

M_{max} = 72.88 kN-m

For A_s = 658 mm^2, d_e = 231 mm, a=48.26mm<51 mm, rectangular section behavior

M_{As} = 65.73 kN-m, P(expected) = 4*M/L(length) = 4*657300/1620 = 161.75 kN

-<u>Rectangular beams</u>:

For A_s = 400 mm^2, d_e = 183 mm, a=91mm

M_{As} = 27.41 kN-m, P(expected) = 4*M/L(length) = 4*27408/1020 = 106.75 kN

Computation of M_u

-<u>For the T-section beams</u>

M_u = C_{cf}*(h-h_f/2) + C_{cw}*(h-a/2) - A_s*f_y*(h-d_e), where a=C_{cw}/(0.85*f'_c*b_w)

For 3 layers of Tonen sheet:

C_{cf} = 0.85*f'_c*(b_f-b_w)*h_f = 222.62 kN

T_{CFRP} = A_{CFRP}*Ft_{CFRP} = 52.02*3480 = 181.03 kN

C_{cw} = (A_s*f_y+T_{FRP})- C_{cf} = 270 kN

a = 123 mm

M_u = 104.5 kN-m, P(expected) = 4*M/L(length) = 4*104500/1620 = 257.15 kN

For 1 sika plate (w = 100 mm):

C_{cf} = 0.85*f'_c*(b_f-b_w)*h_f = 222.62 kN

T_{CFRP} = A_{CFRP}*Ft_{CFRP} = 119*2400 = 285.6 kN

C_{cw} = (A_s*f_y+T_{FRP}) - C_{cf} = 386 kN

a = 176 mm

M_u = 122.47 kN-m, P(expected) = 4*M/L(length) = 4*122470/1620 = 301.35 kN

-For the Rectangular beams: Since no CFRP was use for flexural strengthening, $M_u=M_{As}$ calculated above.

- **Computation of Shear Reinforcement (T-section beams)**

$M_u=P*L/4$, for $M_u=M_{max}$ = 72.88 kN-m, P_u= 179 kN

$V_u=P_u/2$ = 89.7 kN

$V_c = 0.17\sqrt{f'_c}*b*d$ = 19.66 kN, $V_s= V_u-V_c$ = 70 kN

#6 rebar actual f_y = 325 MPa

$A_{v(max)} = V_s*s/(f_y*d)$ = 71 mm^2

$A_{v(used)}$ = 64.5 mm^2

- **Computation of Shear Capacity**

- For the rectangular section

For control beam:

$V_c = 0.17\sqrt{f'_c}*b*d$ = 15.56 kN, P(expected) = V_c*2 = 15.56*2 = 31.14 kN

For beam with CFRP shear reinforcement:

$V_{As} = 0$

$V_{ACFRP} = A_{CFRP}*f_{CFRP}*h/s$, assuming development of full capacity of the CFRP strip in tension

V_{ACFRP} (Tonen) = 2*0.17*3480*254 = 292 kN,

P(expected) = $(V_{ACFRP}+V_c)*2$ = 615.16 kN

V_{ACFRP} (Sika) = 2*29.75*2400*254/75 = 485 kN,

P(expected) = $(V_{ACFRP}+V_c)*2$ = 1001 kN

- For the T section beams

For control beam:

$V_c = 0.17\sqrt{f'_c}*b*d$ = 15.56 kN

$V_{control} = V_{As} + V_c$ = 83.36 kN,

P(expected) = $V_{control}*2$ = 166.71 kN

For beam with CFRP shear reinforcement:

$V_c = 0.17\sqrt{f'_c}*b*d$ = 15.56 kN

$V_{control} = V_{As} + V_c$ = 83.36 kN

$V_{ACFRP} = A_{CFRP}*f_{CFRP}*(h - h_f)/s$, assuming development of full capacity of the CFRP strip in tension.

V_{ACFRP} (Tonen) = 2*0.17*3480*254 = 292 kN,

P(expected) = $(V_{control}+V_{CFRP})*2$ = 745.75 kN

V_{ACFRP} (Sika) = 2*29.75*2400*254/75 = 485 kN,

P(expected) = $(V_{control}+V_{CFRP})*2$ = 1131.75 kN

UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 5: Behavior of Beams Under Cyclic Loading at Low Temperature

by

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Project Advisor: Thomas D. Gillespie, GLCTTR

Report No. UMCEE 98 - 39
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16. Abstract

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:
Volume 1 – Summary Report
Volume 2 – Literature Review
Volume 3 – Behavior of Beams Strengthened for Bending
Volume 4 – Behavior of Beams Strengthened for Shear
Volume 5 – Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications

The part of the investigation dealing with the tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading is the subject of this volume (volume 5). Results are also analyzed, compared, and discussed. Four reinforced concrete beams strengthened with CFRP sheets were designed, prepared and tested under low temperature conditions (-29°C). Two beams were tested monotonically to failure and the other two were tested under high amplitude cyclic load (fatigue load). Parameters investigated were: Low temperature and loading conditions. The four beams were tested under low temperature conditions (-29°C). Two beams were tested under a four-point load configuration, one monotonically and one in cyclic fatigue. These beams, considered to fail in bending, were strengthened with the Sika system. The remaining two beams were tested under three-point load configuration, also one monotonically and one in cyclic fatigue. These beams considered to fail in shear were retrofitted with the Tonen system. The amplitude of the cyclic fatigue load was taken as 10-80% of the failure load from the monotonic test. Conclusions are drawn and some recommendations for design are suggested.

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The opinions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors.

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PREFACE

This project titled: *"Repair and Strengthening of Reinforced Concrete Beams using CFRP Laminates"* is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of the project were:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) glued-on plates for repair and strengthening of concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

The project consisted of 8 tasks as follows:

- A report containing a literature review and a comprehensive synthesis of the latest state of knowledge on the glued -on FRP technique (Task 1);
- Laboratory testing and verification of the selected CFRP glued-on technique according to the proposed experimental program: bending (Task 2), shear (Task 3), freeze-thaw (Task 4), temperature and high cyclic amplitude load (Task 5);
- An interim and final report summarizing the experimental results (Task 6). The interim report will cover the bending and freeze-thaw tests;
- A summary of field specifications and "how to" details for implementation in field applications;
- Guidelines for design based on the experience developed from the experimental work (Task 7);
- Field monitoring of application of the technique to one bridge selected by MDOT (Task 8a);
- Bridge testing before and after application of the glued-on plate (Task 8b to be conducted by professor A. Nowak, U of M)

This report summarizes the experimental program of beams under cycling loading at low temperature as per Task 5.

ABSTRACT

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report is part of a larger investigation on the use of carbon fiber reinforced plastic (CFRP) sheets for repair and strengthening of reinforced and prestressed concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, 2) to synthesize existing knowledge, 3) to identify optimum parameters for successful implementation, 4) to develop procedures for implementation in the field, and 5) to adapt the technique to the specific conditions encountered in the state of Michigan.

The experimental program includes four main parts: 1) tests of RC beams strengthened in bending; 2) tests of RC beams strengthened in shear; 3) freeze-thaw tests of strengthened beams followed by their test in bending; and 4) tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading.

The part of the investigation dealing with the tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading is the subject of this report. Results are also analyzed, compared, and discussed. Four reinforced concrete beams strengthened with CFRP sheets were designed, prepared and tested under low temperature conditions (-29°C). Two beams were tested monotonically to failure and the other two were tested under high amplitude cyclic load (fatigue load). Parameters investigated were: Low temperature and loading conditions. The four beams were tested under low temperature conditions (-29°C). Two beams were tested under a four-point load configuration, one monotonically and one in cyclic fatigue. These beams, considered to fail in bending, were strengthened with the Sika system. The remaining two beams were tested under three-point load configuration, also one monotonically and one in cyclic fatigue. These beams considered to fail in shear were retrofitted with the Tonen system. The amplitude of the cyclic fatigue load was taken as 10-80% of the failure load from the monotonic test. Conclusions are drawn and some recommendations for design are suggested.

The experience gained during this project should contribute to a better understanding of the behavior of these new strengthening systems under different environmental conditions.

EXECUTIVE SUMMARY

This report presents the summary of experimental work, laboratory testing, and analysis of results for task 5 of the current research project which deals with the tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading. Results are also analyzed, compared, and discussed.

Based on the results from the experimental work, the following conclusions were drawn:

1. Prior tests (Task 2) showed that the failure mode of RC beams strengthened with CFRP Sika plates and loaded in monotonic bending at normal room temperature was by delamination of the CFRP plate. The limited tests carried out in this task with low temperature (-29 °C) and cyclic fatigue loading, suggest that the failure mode remains the same, that is by delamination.

2. The strain data of the cyclic fatigue test in bending showed redistribution of strains (thus stresses) in the CFRP plate with an increasing number of cycles. A more uniform strain pattern was achieved suggesting that slow delamination of the plate occurred during cycling. Higher strains at the end of the plate confirmed the extension of delamination toward that section and subsequent delamination failure at that section.

3. Values of the interfacial shear stress from the strains recorded by the gages showed that the interfacial strength at failure was similar for both the monotonically flexure tested beam (1.62 MPa) and the fatigue flexure beam after 155,500 cycles (1.58 MPa).

4. Failure in the shear beam subjected to monotonic loading at -29 °C occurred by shear delamination of the CFRP Tonen sheet followed by shear failure of the concrete. The shear delamination was due to the propagation of a diagonal shear crack within the concrete that extended from the load point to the supports.

5. Failure in the shear beam that was subjected to cyclic fatigue loading at -29 °C was initiated by failure of one of the reinforcing bars in the first layer of steel, and shortly followed by failure of two additional bars. Subsequent analysis suggested that failure of the rebars was by brittle fracture.

6. Increase in the shear strain obtained with cyclic shear loading from the rosette gage placed at midspan along the vertical axis of the beam suggest that some delamination and cracking were occurring at that section. It is likely that a shear failure would have occurred in a manner similar to the monotonically tested beam, should failure of the reinforcing bar not have occurred.

1. INTRODUCTION

This study is part of a research project at the University of Michigan supported by the Michigan Department of Transportation and the Great Lakes Center for Truck and Transit Research. The project title is "Repair and Strengthening of Reinforced and Prestressed Concrete Beams Using Carbon Fiber Reinforced Plastic (CFRP) Glued-on Plates". The study is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of this project are to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, to synthesize existing knowledge and develop procedures for implementation in the field, and to adapt this technique to the specific conditions encountered in the State of Michigan. The project consists of 8 tasks. Tasks 5 deals with the flexural and shear testing of reinforced concrete beams with glued-on CFRP plates subjected to low temperature and high amplitude load. It was of the interest of this research project to test the effectiveness of this strengthening system in cold temperatures as well as fatigue loads, both situations that could be encountered during the service life of a structure strengthened with externally glued-on CFRP plates. This is the subject of this report.

1.1. Organization of this Report

This report presents the summary of experimental work, laboratory testing, and analysis of the results for Task 5 of the current research project: Flexural and shear testing of reinforced concrete beams with glued-on CFRP plates subjected to low temperature and high amplitude cyclic load.

Chapter 2 presents the experimental program. The main experimental parameters, the material properties, fabrication of specimens, and test set-up and instrumentation are described.

Chapter 3 summarizes the procedure followed to test the specimens under low temperature conditions. Design of a controller system is explained.

Chapter 4 presents the results from the bending and shear tests of specimens subjected to low temperature and high amplitude load.

Chapter 5 presents the interpretation and analysis of the experimental results.

Chapter 6 presents the conclusions based on this experimental study.

Chapter 7 provides a list of references.

Chapter 8 includes Appendix A: Design example

2. EXPERIMENTAL PROGRAM

2.1. Parameters of study

Four reinforced concrete beams strengthened with CFRP sheets were designed, prepared and tested under low temperature conditions (-29°C). Two beams were tested monotonically to failure and the other two were tested under high amplitude cyclic load (fatigue load).

Parameters investigated were:
1) Low Temperature. The four beams were tested under low temperature conditions (-29°C).
2) Loading conditions.
   Two beams were tested under a four-point load configuration, one monotonically and one in cyclic fatigue. These beams, considered to fail in bending, were strengthened with the Sika system. The remaining two beams were tested under three-point load configuration, also one monotonically and one in cyclic fatigue. These beams, considered to fail in shear, were retrofitted with the Tonen system.

   The amplitude of the cyclic fatigue load was taken as 10-80% of the failure load from the monotonic test.

Table 2.1. presents the summary of the parameters tested.

Table 2.1 Test Parameters for Low Temperature and Fatigue Load Conditions

Beam Description	Temperature	Type of Loading
Bending Testing	-29°C	Monotonic to failure
Bending Testing	-29°C	High amplitude cycling (10% to 80% of ultimate bending strength)
Shear Testing	-29°C	Monotonic to failure
Shear Testing	-29°C	High amplitude cycling (10% to 80% of ultimate shear strength)

2.2. Material Properties

2.2.1. Concrete

The beams were fabricated in the structural lab of the Department of Civil and Environmental Engineering at the University of Michigan. Concrete was ordered from a ready mix concrete company. Specifications for the mix were provided to the ready mix company: Portland cement type I, sand type 2NS were used as well as a coarse aggregate (source: Stoneco Stone

Co.) with size and gradation corresponding to a 26A and a dilation number (0.0023) that meet MDOT requirements regarding freeze-thaw dilation (MTM 113-97). Two additives were provided: an air entraining agent, in order to obtain a minimum air content of 6.5% $\pm$ 1.5% according to MDOT requirements, and a superplastisizer for better workability of the mix during placement. The specified compressive strength was 34.5 MPa.
The mix proportions of the concrete as provided by the supplier are presented in Table 2.2.

Table 2.2 **Proportions of the Concrete Mix**

Materials	Proportions (Kg/m^3)
Cement Type I	390
Sand (2NS)	759
Limestone (26A)	1020
Water	148
Superplastisizer (Daracem 55)	1528 ml/m^3
Air Entraining Agent (Darex II)	383 ml/m^3

The properties of the fresh concrete were as follows:
- Air content. The volume of air contained in the concrete mix was measured using a roll-a-meter. The average value obtained was 5%, which falls within the admissible range.
- Slump > 254 mm (note: this high value of slump may be caused by the addition of excessive superplastizicer by the ready mix company. As indicated below, the compressive strength of the concrete was below the target value of 34.48 MPa)

The actual compressive strength was obtained from cylinders (102 mm diameter, 203 mm height) tested at the time of testing of each beam specimen. Average values (of two cylinders tested) are presented in Table 2.3.

Table 2.3 **Compression Test Results**

Date	Description	Age at testing	Compressive Strength (MPa)
May 20, 1998	Pouring of Concrete	-	-
May 28, 1998	Application CFRP	7 days	19.65
June 4, 1998	Monotonic shear	15 days	23.24
June 5-8, 1998	Fatigue shear	15 days	23.24
June 11, 1998	Monotonic flexure	21 days	24.13
July 23–August 3, 1998	Fatigue flexure	60 days	30.49

2.2.2. Reinforcing bars

The steel reinforcing bars were Grade 420 with minimum yield strength of 420 MPa and a tensile modulus of 200 GPa. Two diameter were used, 10 mm (No. 10) and 13 mm (No.13).

2.2.3. Strengthening Systems

2.2.3.1. Tonen System

The strengthening system was supplied by Master Builders. Commercial name: MBrace Composite Strengthening System. It has five components:

- MBrace Primer
- MBrace Putty filler (not used)
- MBrace Saturant Resin
- MBrace Fiber Reinforcement (MBrace CF130 Carbon fiber system)
- MBrace Topcoat (not used)

MBrace Putty filler was not used since it is intended to be used to patch cracks and the concrete used did not required this surface preparation. MBrace Topcoat is an optional finishing layer for painting appearance and UV protection. Since the testing of the beams was to be performed indoors, this finishing was not used on this experimental program.

Typical Properties of MBrace CF 130 (provided by the supplier):
Fiber Reinforcement: Carbon Fiber, High Tensile
Fiber Density: 1.82 g/cm^3
Fiber Modulus: 2.35 x10^6 kg-force/cm^2
Fiber Areal Weight Density: 300g/m^2
Sheet Width: 50 cm
Tensile Strength: 590 kg-f/cm-sheet width
35,500 kg-f/cm^2
Tensile Modulus: 38,800 kg-f/cm-sheet width
2.35 x10^5 kg-f/cm^2
Design Thickness: 0.165 mm/ply
Tensile Elongation at ultimate: 1.5%

Typical Properties of MBrace Saturant (provided by the supplier):

Volatile Organic Compounds: 20 g/liter
Flash Point: 72°C
Mixed Viscosity @ 20°C: 1,600 cps
Color: Blue
Weight/Gallon: 1.04±0.024 kg/L
Shelf Life @ 20°C: 18 months
Flexural Strength: 43 MPa
Tensile Strength: 78 MPa
Compressive Strength: 88 MPa

Work Time @ 20°C: 30 minutes

Typical Properties of MBrace Primer (provided by the supplier):
Generic type: Amine-cured liquid epoxy
Solids content: 100%
Color: clear Amber
Weight/Gallon: Part A 1139g/L
Part B 996 g/L
Tensile Strength: 13 to 15.8 MPa
Tensile Modulus (Tangent): 689 to 826.8 MPa
Tensile elongation: 20-30%
Tensile bond strength (steel): 17 MPa
Work Time @ 20°C: 45 hours

2.2.3.2. Sika System

The Sika Company provided the strengthening system. Components of this system are:
- Sika Carbodur CFRP (Carbon fiber laminate strips).
- Sikadur 30 (epoxy adhesive).

Typical Properties of Sika CFRP Strips (provided by the supplier):
Tensile Strength: 2,400 MPa
Modulus of Elasticity: 150 x10^3 MPa
Density: 1.6 g/cm^3
Thickness: 1.2 mm
Sheet width: 50 or 80 mm
Elongation at ultimate: 1.4%

Typical Properties of Sikadur 30(provided by the supplier):
Application Temperature: 18-30°C
Pot Life @23°C: 70 min
Compressive Strength (14 day) > 58.6 MPa
Shear Strength: 24.8 MPa
Tensile Strength @7 day: 24.8 MPa
Elongation at break: 1%

2.3. Design and Fabrication of Specimens

2.3.1. Flexural Beams

The issue of cooling the specimens during testing was addressed before design of beams. A square hollow covered section was considered to be the most efficient for achieving a uniform low temperature. A cross section of 203 mm x 203 mm with a centered hollow core of 76 mm diameter was chosen. The cooling process was provided by injection of liquid nitrogen in the hollow section. Specific details of the design of the cooling system will be provided in the next section.

The design of the specimens for flexural tests was based on the results from the previous bending tests (Task 2). The span to depth ratio (l/d) was intended to be similar to the one used for the previous tests, l/d=12.58. The beams had a length of 1981 mm. Longitudinal steel was designed such as for a compressive strength of 34.5 MPa, the reinforcement ratio will be 2/3 of ρ_{max}. At the time of testing of each specimen, it was found that the compressive strength of the concrete was different than the specified value. Corrected steel reinforcement ratios as a percent of ρ_{max} are presented in Table 2.4. It was assumed that the aluminum tube used to cast the hollow section did not provide any contribution to the flexural strength of the specimen and did not affect its strength.

The test beams had four longitudinal reinforcing bars, placed in two rows. The lower row had two No. 13 bars centered at 38 mm from the bottom fiber. The upper row had one bar No. 13 and one bar No. 10 with a center to center distance of 38 mm from the lower row. Two additional No. 10 bars were provided in a top layer located 38 mm from the top fiber of concrete. To ensure flexural failure with the addition of the CFRP laminate, sufficient stirrups were provided (see Appendix A). The cross sections of these beams as well as load set-up are presented in Figure 2.1.

Wood molds were utilized for the fabrication of the specimens. Thermocouples were placed at different locations in order to record the temperature during the testing. A more detailed description of the location of thermocouples is given in the next section.

2.3.2. Shear Beams

Shear beams had the same cross section as the flexure beams. Longitudinal steel was also placed in the same way as for the flexure beams (see previous section). No stirrups were provided in order to measure the contribution of the CFRP system to shear resistance. It was assumed that the aluminum tube used to cast the hollow section did not affect the shear strength of the specimen.

The shear span to depth ratio (a/d) of 2.5 was the same as that used in the previous shear tests (Task 3). The total length of the shear beams was 1016 mm. Load set-up for the shear specimens is presented in Figure 2.1.

2.3.3. CFRP Application

All surfaces of the specimens that were going to receive a CFRP sheet were ground with a disk grinder. The concrete surface was ground enough to remove laitance and show the open texture of the aggregates. After grinding the dust was removed by brushing and vacuum cleaning.

Table 2.4. Test parameters for bending and shear tests

Beam No.	Test parameter	Reinforcement ratio, ρ	A_s (used) mm^2	Effective depth (mm)	CFRP reinforcement
1	Monotonic shear	$0.89\ \rho_{max}$	3#10 1#13 $A_s = 458$	$d_s = 141$ $d_f = 203$	Shear reinf: 1 layer Tonen sheet
2	Fatigue shear	$0.89\ \rho_{max}$			
3	Monotonic flexure	$0.85\ \rho_{max}$			Flexural reinf: 1 layer Sika plate
4	Fatigue flexure	$0.70\ \rho_{max}$			

The Tonen sheets and the Sika plate were cut to proper lengths using a sharp blade and a disk cutter, respectively. The adhesive components were mixed according to the technical data sheet provided by the respective system supplier. Application of each strengthening system followed the supplier recommendations.

The reinforced concrete beams were not precracked before the application of the CFRP material.

A Sika CFRP plate of 102 mm width and 1753 mm length was placed at the bottom side of the flexural beam specimens (see Figures 2.1 and 2.2). The width and length of the CFRP plate were calculated in order to obtain the maximum strengthening. A summary of these preliminary calculations is presented in Appendix A.

For the shear beams, CFRP Tonen sheets were used as the only shear reinforcement. One layer of CFRP sheet was wrapped around the lateral sides (U-shape) of the beam along its total span length (711 mm) as shown in Figures 2.1 and 2.2. Two Tonen sheets of 432 mm width (77 mm overlap at the midspan) and 609 mm length were used on each specimen. The direction of the fibers was perpendicular to the longitudinal axis of the beams.

2.4. Test Set-Up and Instrumentation

A computer based data acquisition system (Megadack System) was used to measure load and displacement of the machine load cell and machine LVDT, as well as strains from the CFRP sheets and plates.

For the flexure tests, six strain gages were placed along the length of the CFRP plate. Two strain gages were placed at midspan (M1, M2), two strain gages were placed at about one third of the span (R2 on the right side and L2 on the left side), and two strain gages were placed at 267 mm from both ends of the CFRP plate (R1 on the right side and L1 on the left side), see Figure 2.2.

For the shear tests, six strain gages in a rosette configuration were placed on one of the lateral sides of each beam. Three rosette strain gages were placed in a vertical arrangement at 178 mm from each end of the CFRP wrap (for the right side: TR top right, MR middle right, BR bottom right; for the left side TL top left, ML middle left, BL bottom left). Location and numbering of the strain gages for each particular test are shown in Figure 2.2.

The two beams that were tested monotonically to failure were loaded using displacement control at a loading rate of 0.13 mm/second. The Instron machine load cell used has a capacity of 450 kN.

The two beams that were tested under fatigue were loaded using load control at a frequency of 3 cycles/second. The target range of testing was 10-80% of the ultimate load from the monotonic test.

Due to safety measures of the structure lab, the fatigue tests had to be stopped at the end of each testing day. The fatigue test of the shear beam was completed in two days. The fatigue test of the bending beam was completed in three days. At the beginning of the third day it was decided to increment the loading range to 10-90%. At the end of the day, and due to the fact that no change was observed in the behavior of the beam, the range was increased to 10-100% during 10,000 cycles. Finally, the beam was tested monotonically to failure under displacement control. Table 2.5 presents a summary of the testing stages for both fatigue tests.

Table 2.5 Summary of the Fatigue Tests Procedure

Test	Cycles (accumulative)	Date	Cycle
Shear Fatigue	0-28902	June 5, 1998	10-80%
	28903-43083	June 8, 1998	10-80%
Bending Fatigue	0-30000	July 23, 1998	10-80%
	30001-85000	July 24, 1998	10-80%
	85001-145999	August 3, 1998	10-90%
	146000-15499	August 3, 1998	10-100%
	155500	August 3, 1998	Monotonic to failure

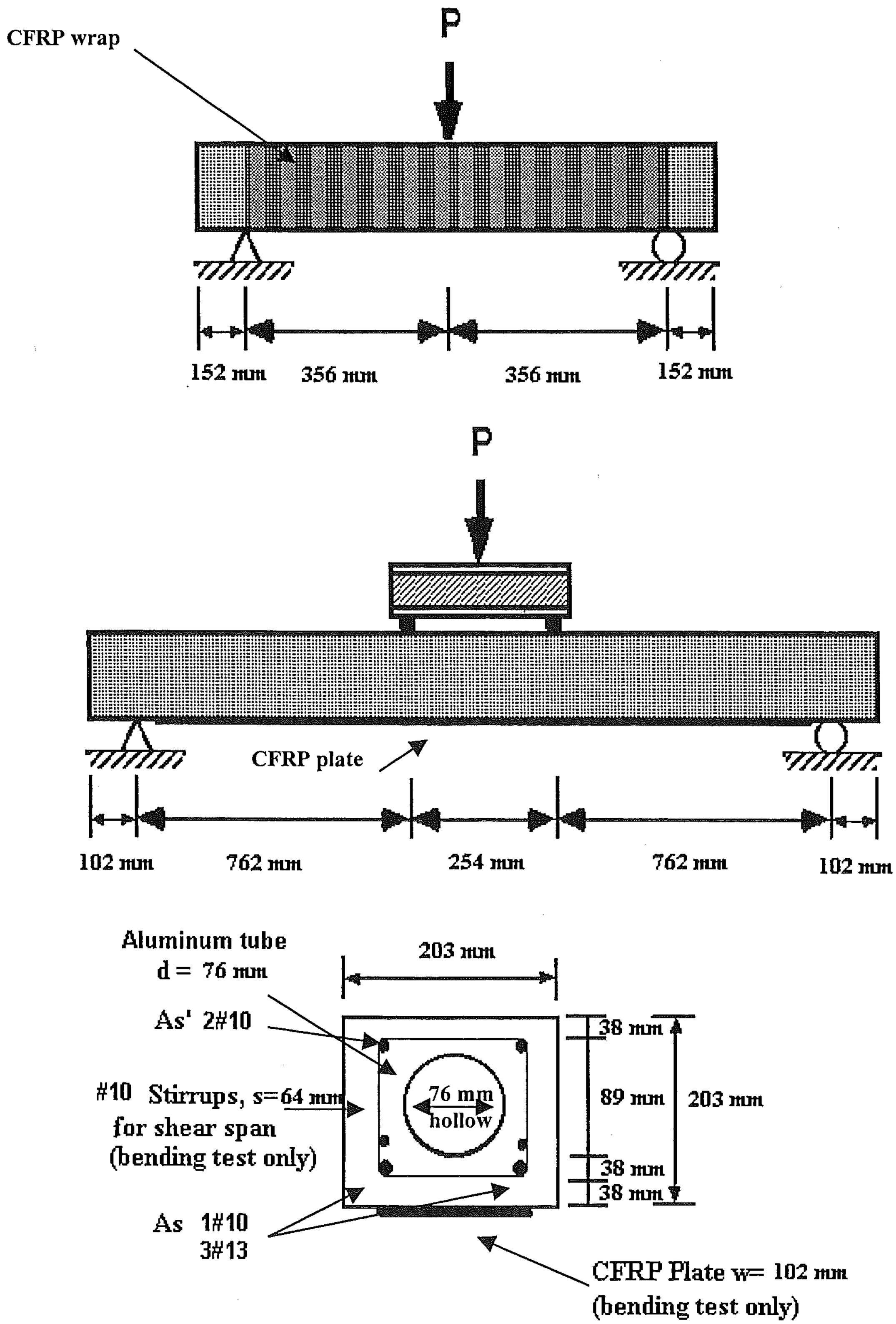
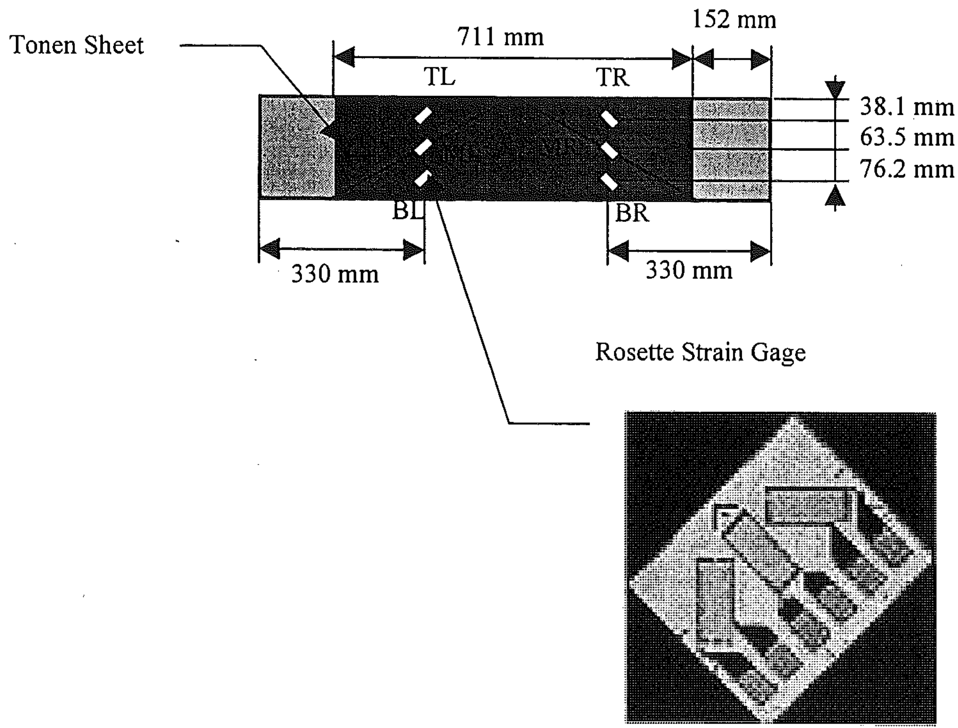
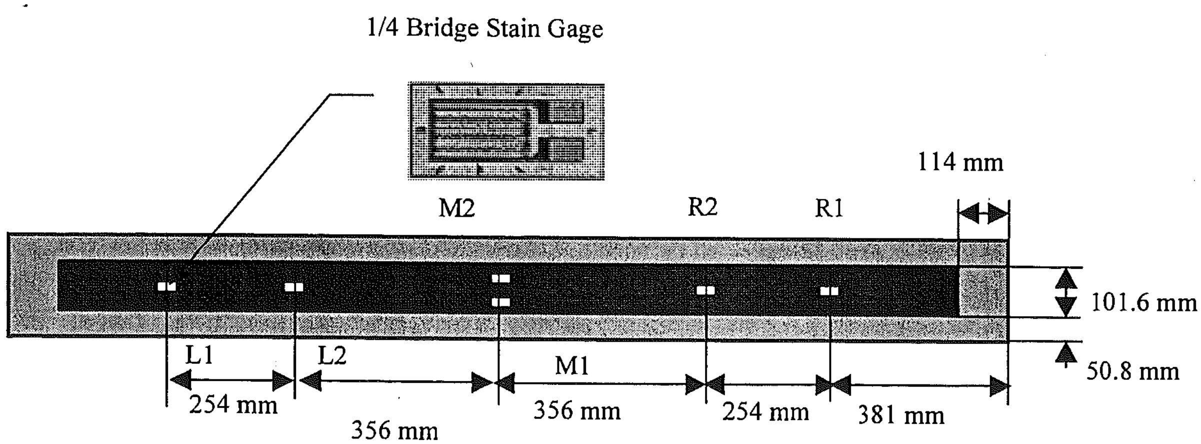


Figure 2.1 Cross Section and Load Set-Up for Shear (top) and Bending (bottom) Test



Shear Specimen Strain Gage Layout



Bending Specimen Strain Gage Layout

Figure 2.2 Strain Gage Layout for the Shear and Bending Tests

3. DESIGN OF TEMPERATURE CONTROL SYSTEM

A temperature control system was designed to regulate the internal specimen temperature. The desired nominal temperature of specimens during testing specified by the project proposal was -29 degrees Celsius. Cooling was achieved by delivering liquid nitrogen into the core of hollow beam specimens, as shown in Figure 3.1. This concept was developed and implemented. The temperature control system instrumentation used to carry out the experiment is described in this section. A brief analysis and discussion of the performance of the cooling system is also included.

3.1. Liquid Nitrogen Cooling System

3.1.1. Specimen Cooling Apparatus

After preparing steel reinforcing cages, each specimen was fitted with a 76 mm diameter aluminum tube (see Figures 3.1 and 3.2). The tubes ran along the long axis of the beam and were held in place with steel wire. The tubes functioned as coolant reservoirs.

The coolant selected was Liquid Nitrogen (N_2). Although the boiling point of liquid nitrogen is much lower than the required temperature of –29 degrees Celsius, the extreme cold temperature allowed quick and efficient cooling of the specimens. The nitrogen was delivered through 12.7 mm diameter soft copper tubes. Several holes were drilled in the copper tubes to distribute the liquid nitrogen into the specimen. Both ends of the aluminum reservoir were capped with plywood plugs and 12.7 mm of concrete. Inlet and outlet plugs in these caps allowed for adding of liquid nitrogen and venting of excess nitrogen gasses. Two inlet tubes were used for the bending specimens, one at each end, and one inlet tube was used for the shear specimens because of the short span. The cooling system configurations for bending specimens and shear specimens are shown in Figures 3.3 and 3.4 respectively.

The flow of nitrogen into the specimen was controlled using a cryogenic rated solenoid valve. The solenoid valve had two states, open and closed. A duty cycle was established to control the flow rate of liquid nitrogen into the system.

3.1.2. Automatic Liquid Nitrogen Delivery System

The primary temperature control mechanism was the cryogenic rated solenoid valve mentioned in the above section. This valve was an integral part of the closed loop control system that regulated the specimen temperature. The system was comprised of three parts: a type "T" thermocouple, a computerized Proportional Integral Derivative (PID) controller, and a valve actuator, see Figure 3.5.

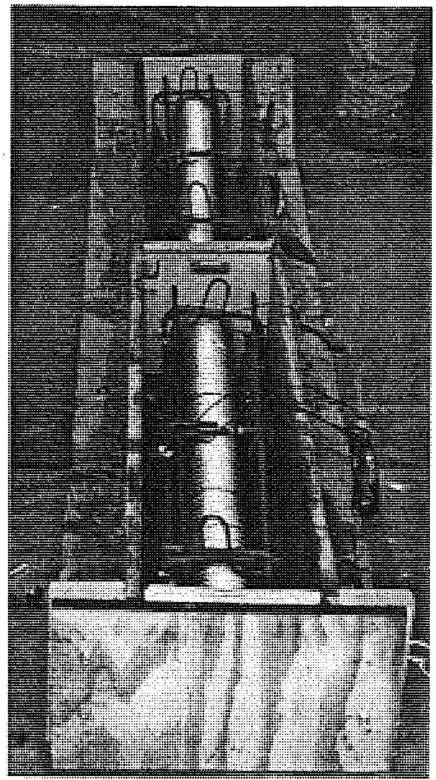


Figure 3.1 Photograph of the Hollow Shear Specimens before Concrete Pouring

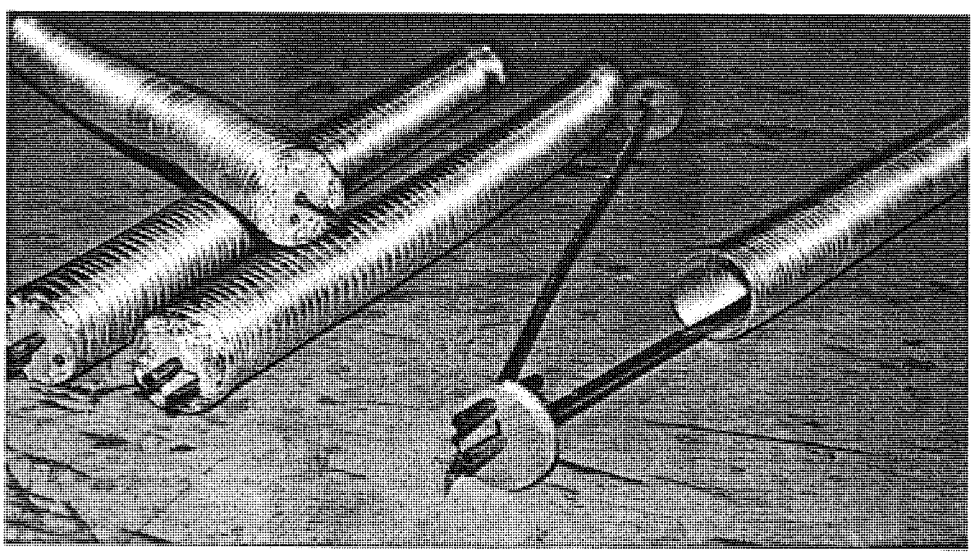


Figure 3.2 Photograph of Coolant Tubes

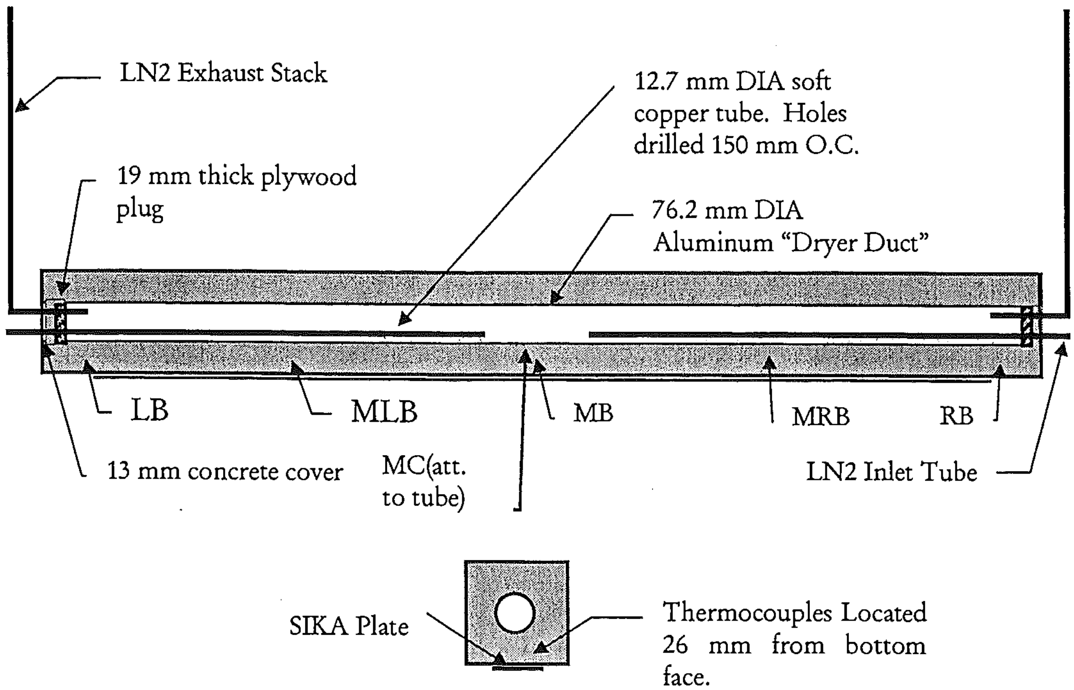


Figure 3.3 Flexural Specimen Cooling System.

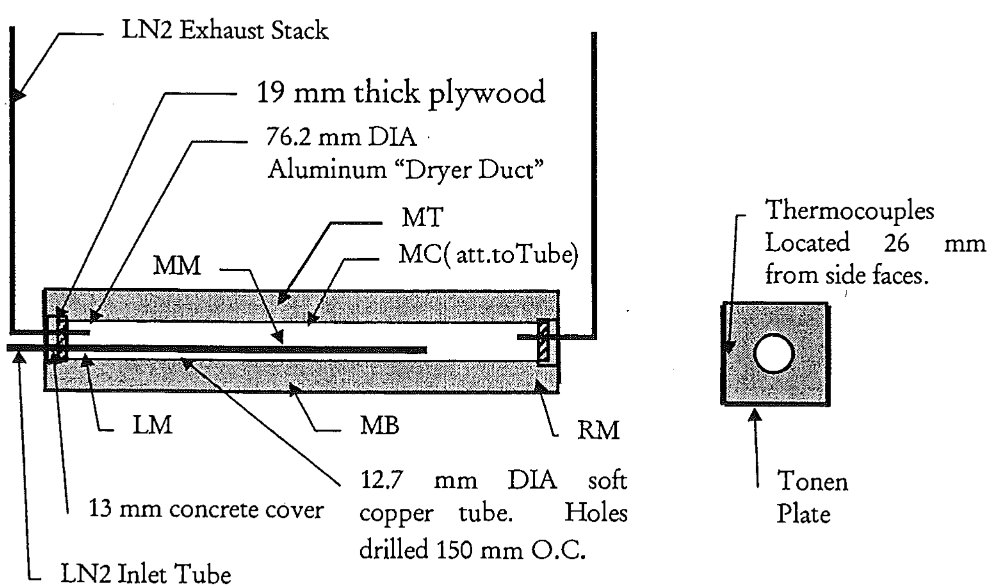


Figure 3.4 Shear Specimen Cooling System.

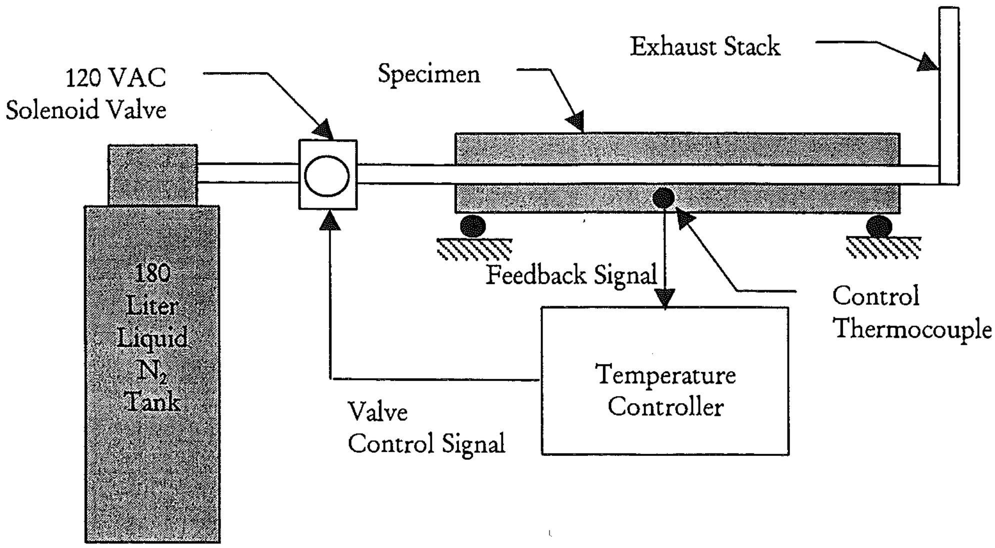


Figure 3.5 Temperature Control Loop

Several thermocouples were embedded in each specimen so differences in the temperature throughout the beam could be measured. The following notation was used to label the thermocouples: LB (left bottom), MLB (middle left bottom), MRB (middle right bottom), RB (right bottom), LM (left middle), RM (right middle) and MC (middle center). All thermocouples were located at a depth of 26 mm from the exterior surface of the beam with the exception of thermocouple MC (middle center). This thermocouple was mounted directly on the cooling tube. The locations of the thermocouples are shown in figures 3.3 and 3.4. One of these thermocouples fed temperature data to the computerized controller. The controller compared this data to user defined target temperature (set-point) and sent an appropriate command signal to the valve. The locations of the thermocouples are shown in Figures 3.3 and 3.4. The center bottom (MB) thermocouple was used for control of the bending specimen temperature because the area of interest was the interface between the CFRP and the concrete at the bottom of the beam. The center-side face (MM) thermocouple was used for the shear specimen because we were interested in the interface between the CFRP and the side surface of the beam.

<u>3.1.3. *Insulation*</u>

A foam insulation system was constructed to increase cooling efficiency and to decrease the thermal gradient between the core and outside surfaces of the specimens. The exterior surfaces of each specimen were fitted with a 25.4 mm thick layer foam insulation. The joints were sealed using spray foam insulation. Notches were cut in the insulation at 100 mm intervals to prevent the foam from having any effects on the strength of the beam. In addition to insulating the specimen, the external coolant tubes were wrapped with standard foam pipe insulation.

After the insulation system was installed, a significant improvement in cooling performance was measured.

3.1.4. Performance

The control of temperature in the area of interest (center bottom for flexure and center side for shear) was good. The coolant "pooled" at the extreme ends of the specimens causing lower temperatures in these regions. To decrease this effect, valves were placed on each exhaust stack to balance the flow of exhaust between the two sides of the specimens. Table 3.1 summarizes the average temperatures of the specimens at the controlled point during testing.

Table 3.1 **Control Thermocouples and Average Temperatures**

Specimen	Controlling Thermocouple.	Temperature Range	Average Temperature
Shear Monotonic	MM (Middle- Side Face)	-31.1 to -23.3 ° C	-27.1° C
Shear Cyclic	MM (Middle-Side Face)	-31.6 to -24.5 ° C	-28.9° C
Flexural Monotonic	MB (Middle-Bottom)	-30 to -28.9 ° C	-29.5° C
Flexural Cyclic	MB (Middle-Bottom)	-28.9 -27.8 to ° C	-28.3° C

Table 3.1. shows that the liquid nitrogen cooling system effectively cooled the beam specimens to the levels specified by the experimental program. Although the temperature was not one hundred percent uniform across the length of the beam, the temperature at the critical areas of the beam was controlled with a high degree of accuracy (+/- 5° C). For the experimental program, it was decided the performance of the system was acceptable.

4. TEST RESULTS

Table 4.1 presents a summary of the test results from the 4 beams tested, along with the predicted failure loads. Peak load, maximum shear force and maximum moment at failure and failure modes are reported for each beam. Figures 4.1 to 4.4 show the load versus deflection curves for each specimen. For the specimens tested under fatigue load, selected cycles are plotted. In order to visualize these graphs, the origin of the x axis was shifted by 2.5 mm for each of the fatigue curves.

Table 4.1 Summary of the Test Results

Beam	Parameter	Predicted load (KN)	Peak Load (KN) Range (%)	Max. Shear (KN)	Max. Moment (KN-m)	Type of Failure
1	Shear Monotonic	246	206	103	37	Concrete shear crack from load point to supports. Debonding of the CFRP sheet along the major crack.
2	Shear Fatigue	N.A.	20.6-165 10-80%	82.4	29	Total # cycles = 43083 Fatigue failure of the bottom layer of steel. A vertical crack of the entire concrete section and rupture of the CFRP sheet under the load point.
3	Flexure Monotonic	93	131	65.7	50	Partial debonding of the CFRP plate. Interfacial delamination of the CFRP plate at the extreme end.
4	Flexure Fatigue	N.A.	13.1-105 10-80% *9.4-75%	52.6	40	# of cycles = 85,000
		N.A.	13.1-118 10-90% *9.4-85%	59.1	45	# of cycles = 61,000 Total # cycles = 146,000
		N.A.	13.1-131 10-100% *9.4-94%	65.7	50	# of cycles = 9,500 Total # cycles = 155,500
		93	Monotonic 139.15	69.6	53	Total # cycles = 155,500 Partial debonding of the CFRP plate. Interfacial delamination of the CFRP plate at the extreme end. Crushing of concrete at the top layer midspan.

* Modified range according to maximum load of the monotonic test of the "fatigue" specimen.

N.A. Not Applicable.

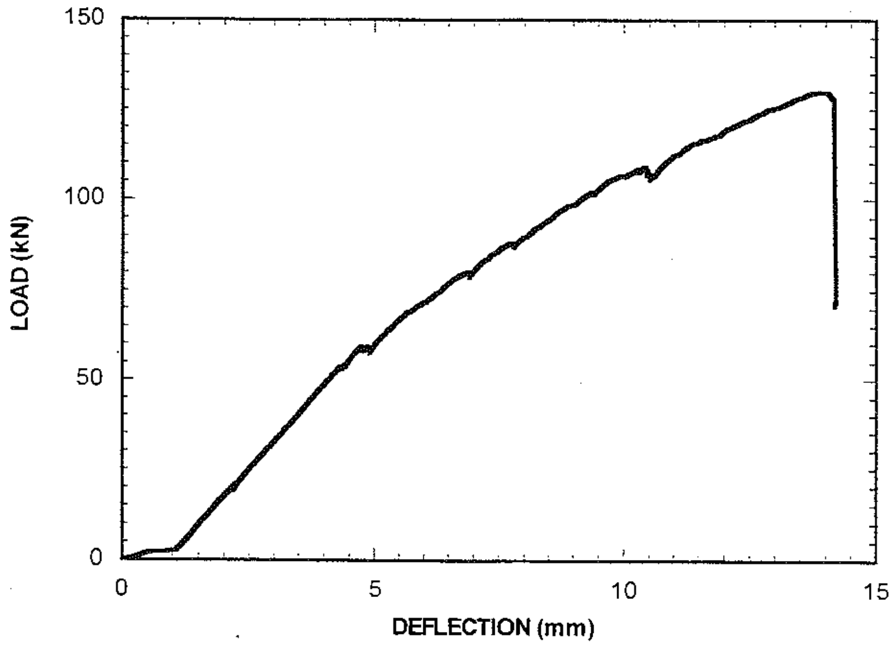


Figure 4.1 Load versus Deflection Curve for Monotonic Bending Test

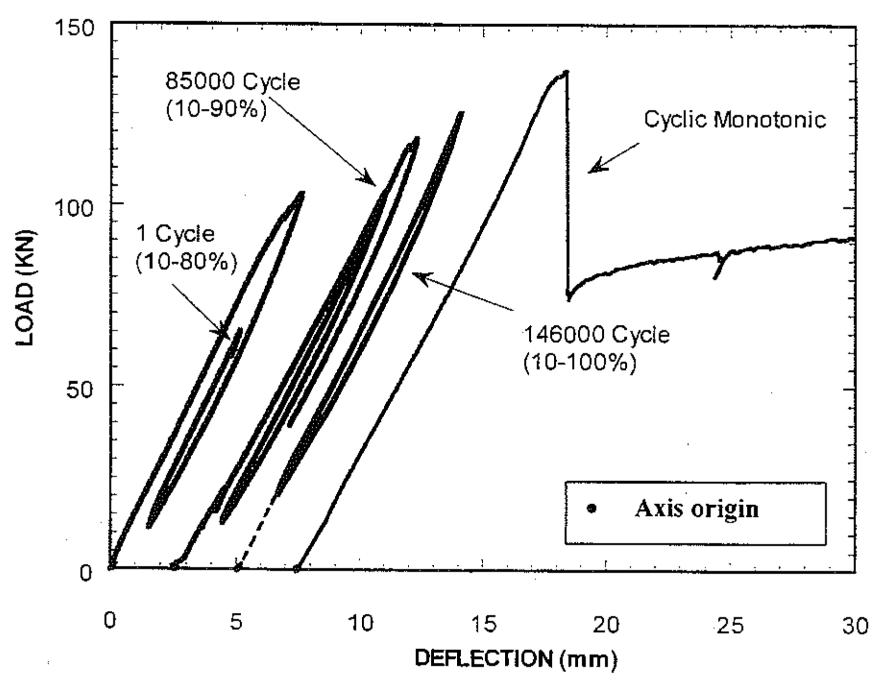


Figure 4.2 Load versus Deflection Curves for Fatigue Bending Test

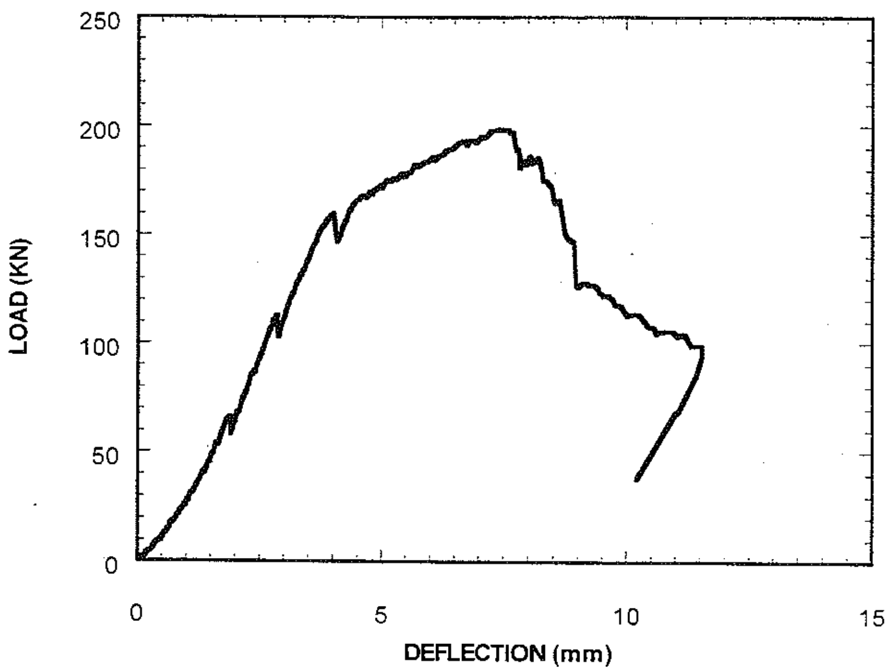


Figure 4.3 Load versus Deflection Curve for Monotonic Shear Test

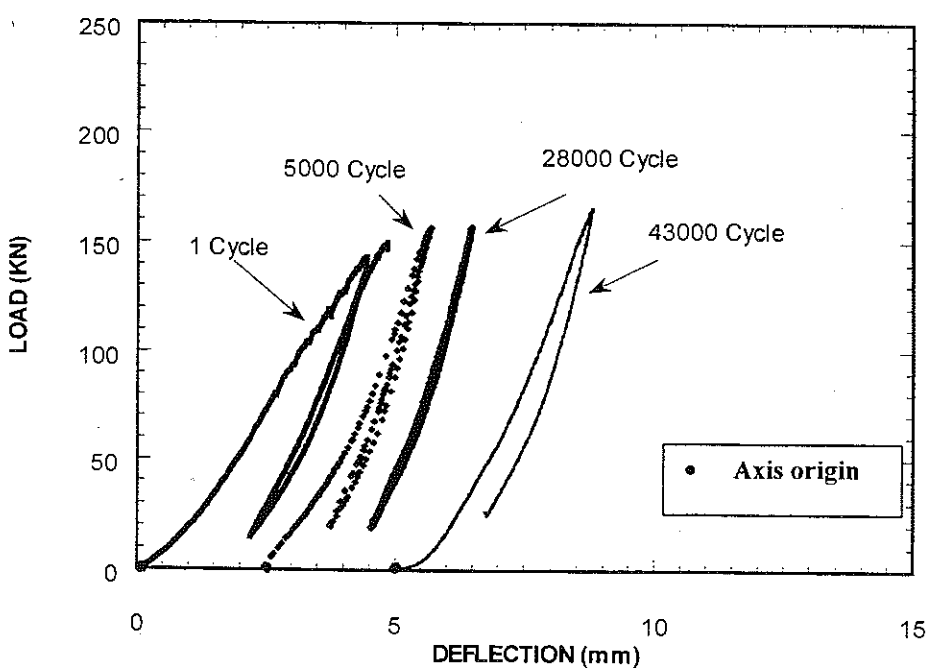


Figure 4.4 Load versus Deflection Curves for Fatigue ShearTest

5. ANALYSIS AND INTERPRETATION

5.1. Steel Reinforcement Ratio

Since the concrete compressive strength of the specimens at the day of testing varied from the target value of 34.5 MPa, the steel reinforcement ratio also varied from the value of 2/3 of ρ_{max}. Table 5.1 shows the new value of the steel reinforcement ratio.

Table 5.1 Summary of the Test Results

Beam Parameter	Day of Testing	f'c (MPa)	Steel Reinf. Ratio
Shear Monotonic	15 days	23.24	0.89 ρ_{max}
Shear Fatigue	15 days	23.24	0.89 ρ_{max}
Flexure Monotonic	21 days	24.13	0.85 ρ_{max}
Flexure Fatigue	60 days	30.49	0.70 ρ_{max}

As can be seen from Table 5.1, the steel reinforcement ratio for all specimens is higher than the value of 2/3 ρ_{max}. However, since it doesn't exceed the value of ρ_{max}, the specimens satisfy AASTHO requirements.

For the flexure specimens, the difference in concrete compressive strength between the date of monotonic testing and fatigue testing was 26%, while the difference between the maximum failure load between the monotonic and the fatigue-monotonic test was 6%. It can be concluded that this variation of the compressive strength does not affect significantly the strength of the strengthened beam.

5.2. Failure Modes

5.2.1. Flexure Test

As was observed in the bending tests of Task 2, beams reinforced with Sika plates failed in all cases by partial debonding of the CFRP plate. For the current flexure tests, failure was also by partial debonding of the CFRP Sika plate. This type of failure has been associated with interfacial shear failure between the surface of the concrete and the CFRP plate. The epoxy adhesive between them tore out the concrete just above the interface epoxy/concrete. The debonding length for both cases was around 1060 mm (60% of the bonded CFRP length), extended from the zone of maximum moment to one of the ends of the CFRP plate. There was no evidence of variation of the failure mode due to the effect of the fatigue cycles. It was also found that at the debonded end of the CFRP plate, interlaminar failure (failure within the laminate) was also present. It is presumed that the release of energy due to debonding may have contributed to this type of failure at the end of the CFRP plate. A closer observation of this phenomenon revealed that only a first layer of single fibers was kept bonded to the adhesive. Therefore this type of failure can still be considered as interface failure (Concrete-epoxy interface).

Figure 5.1 and 5.2 show photographs of both the debonding and the interfacial failure for the monotonic flexure test. In this test, debonding seems to have started at the left shear span of the beam next to the left point load. Figure 5.3 shows the load-strain curves. Strain gage L1 was reported damaged and did not recorded any data. Strains at strain gage L2 significantly decreased at about 110 KN reflecting the occurrence of a debonding process on the proximity of its location. A redistribution of the stresses happens at this level showing an increase in the strain rate for the remaining strain gages.

The flexural beam subjected to cyclic fatigue loading also failed by partial debonding of the Sika CFRP plate. After 155,500 cycles, the beam was tested under monotonic load up to failure. As shown in Figure 5.4 and 5.5 the same type of failure observed for the monotonically tested beam was also observed for this beam. Debonding process also seems to have started on the right shear span. The length of the interlaminar failure within the laminate was 3 times larger (300 mm) than the one obtained from the monotonic test. Note that the total debonding length was about 1060 mm for both cases. The damage of the CFRP plate is more evident, several layers of fibers were separated from the original plate, see Figure 5.6. Figure 5.7 shows the load-strain curves for the monotonic loading test after cycling. Strain gage R1 shows increments of strains up to the failure load, after that point, the strain level registered was the lower of all, indicating that debonding has occurred on this side of the beam. For strain gage R1 the strain rate was higher than for L1. This presence of higher end plate stresses is also related with the presence of the interlaminar failure at this location. Strain gage R2 failed after the maximum load. All the other strain gages also registered a drop of strain at failure.

5.2.2. Shear Test

The monotonic shear beam failed by propagation of a major diagonal crack (shear failure) from the load point to one of the supports (left side). Debonding of the CFRP sheet occurred along this crack due to the relative displacement of the two concrete blocks. Figure 5.8 and 5.9 show photographs of this type of failure. The strain gage rosettes placed on the surface of the Tonen CFRP sheet registered a very small amount of strain. It was also found that the type of epoxy used to glue the rosettes to the surface of the CFRP lost its full bonding properties with the low temperature. Therefore some of the rosettes used did not registered strain. Figure 5.10 shows same load versus shear strain curves for this test. It can be seen that the strain gages on the left side registered significant jumps. These "jumps" are associated to propagation of the shear crack along the concrete beam and the corresponding debonding of the CFRP sheet.

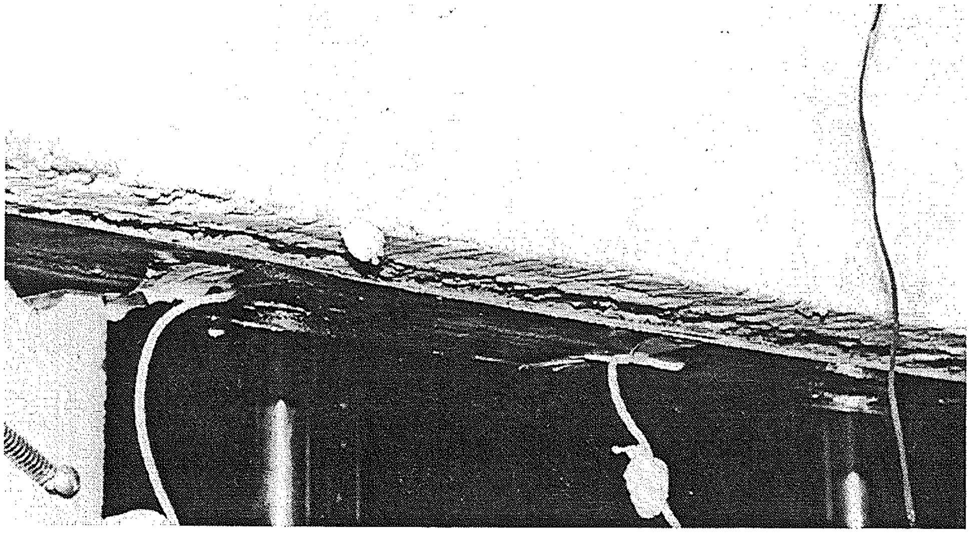


Figure 5.1 Interfacial Failure of the Monotonic Bending Specimen

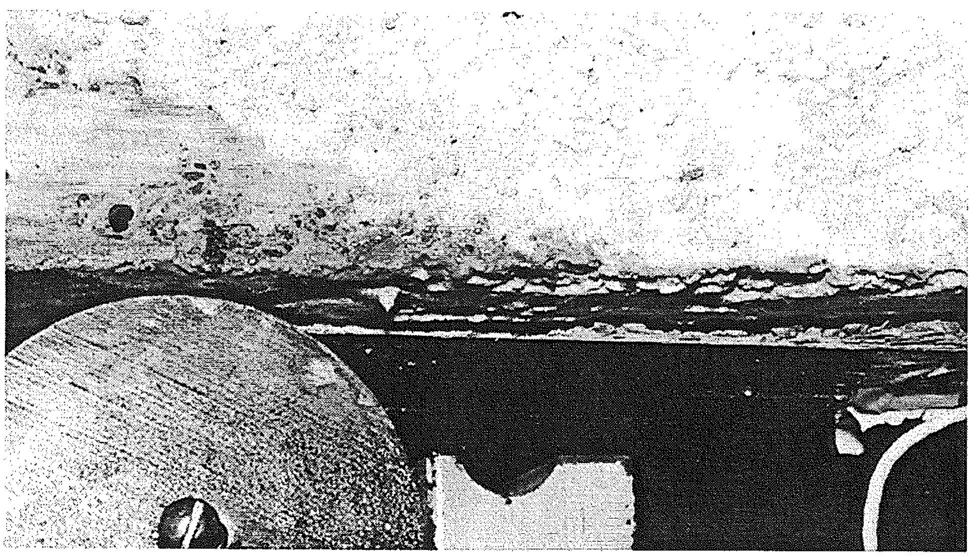


Figure 5.2 Detail of the Interfacial Failure of the Monotonic Bending Specimen

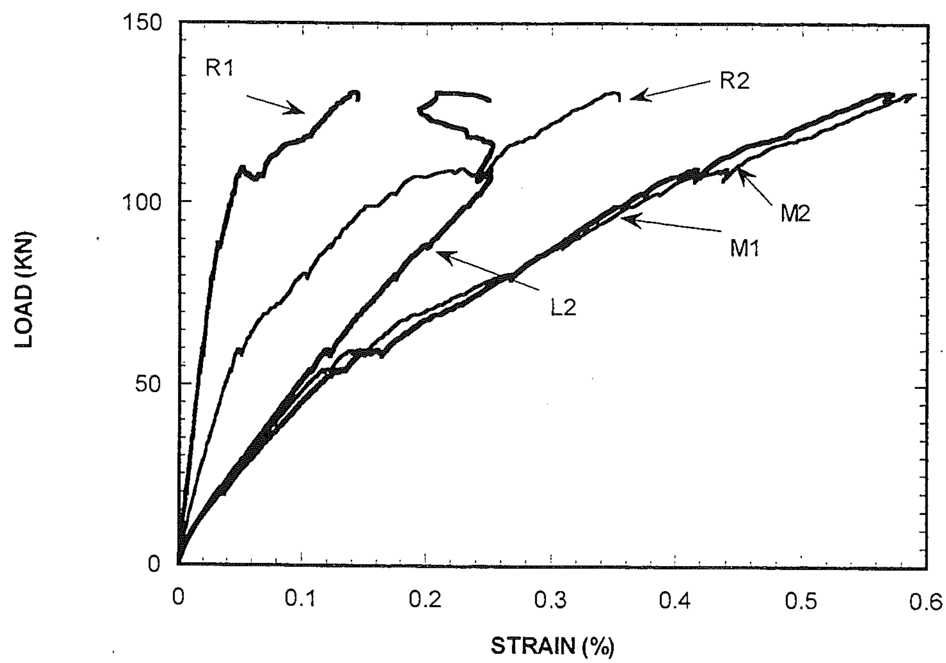


Figure 5.3 Load versus Strain Curves for Monotonic FlexureTest

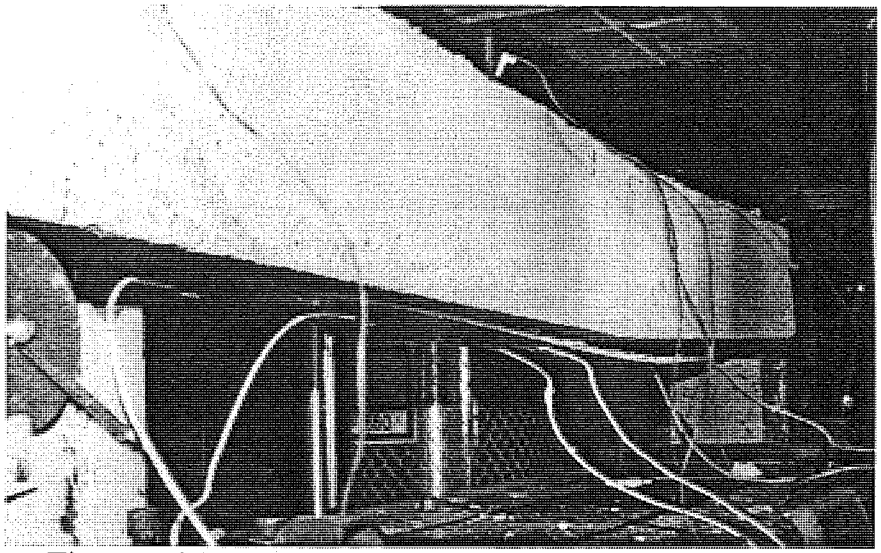


Figure 5.4 Interfacial Failure of the Fatigue Flexure Test

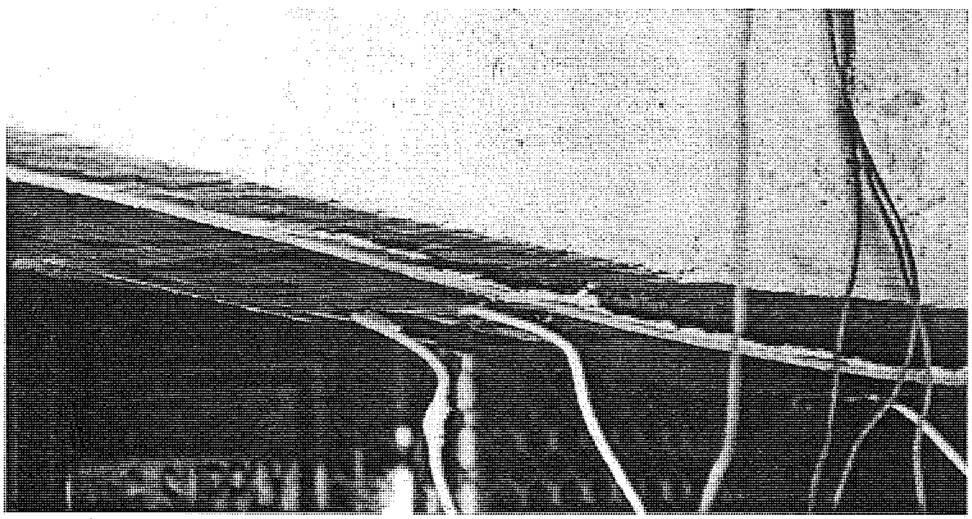


Figure 5.5. Detail of the Interfacial Failure (Fatigue Flexure)

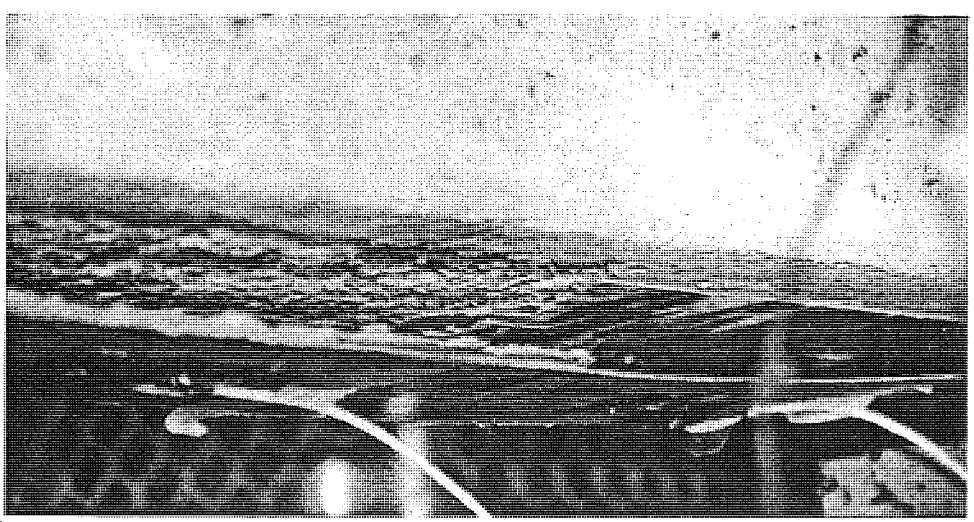


Figure 5.6 Interlaminar Failure at End of the Sika Plate (Fatigue Flexure)

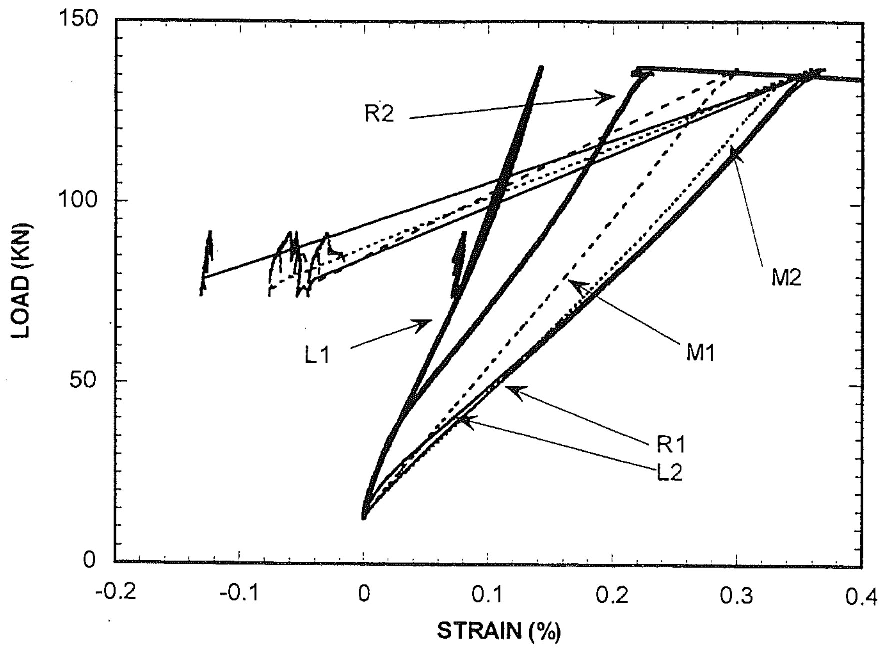


Figure 5.7 Load versus Strain Curves for Monotonic after Cycling Flexure Test

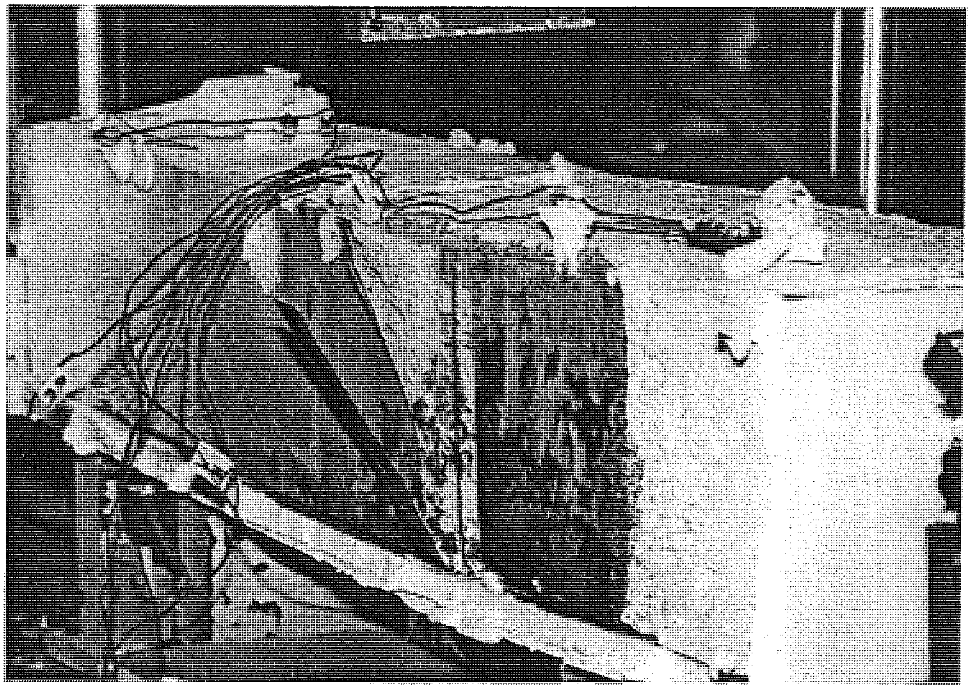


Figure 5.8 Shear Failure and Debonding of the Tonen CFRP sheet (Shear Monotonic)

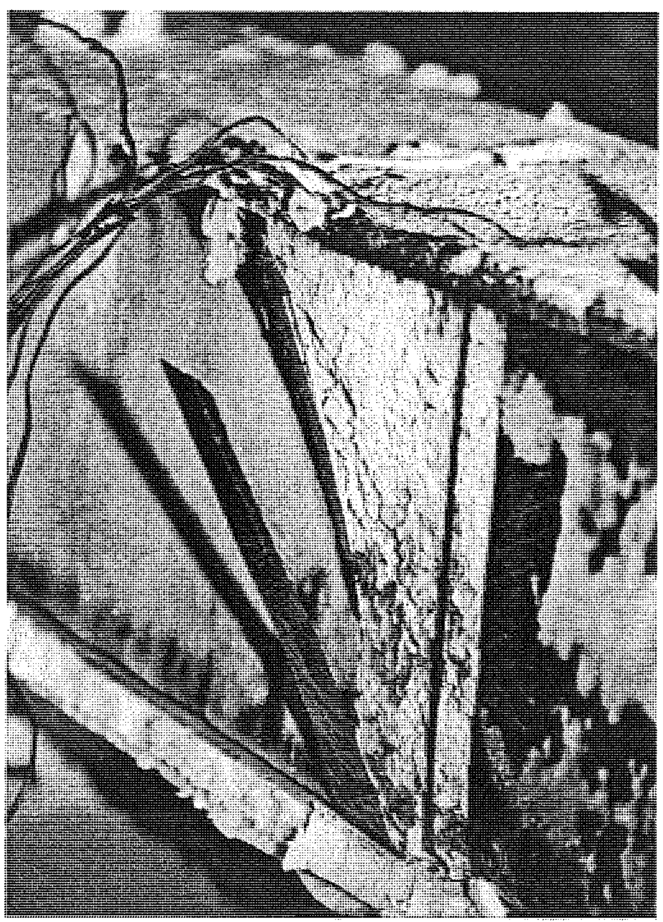


Figure 5.9 Detail of the Debonded Area (Shear Monotonic)

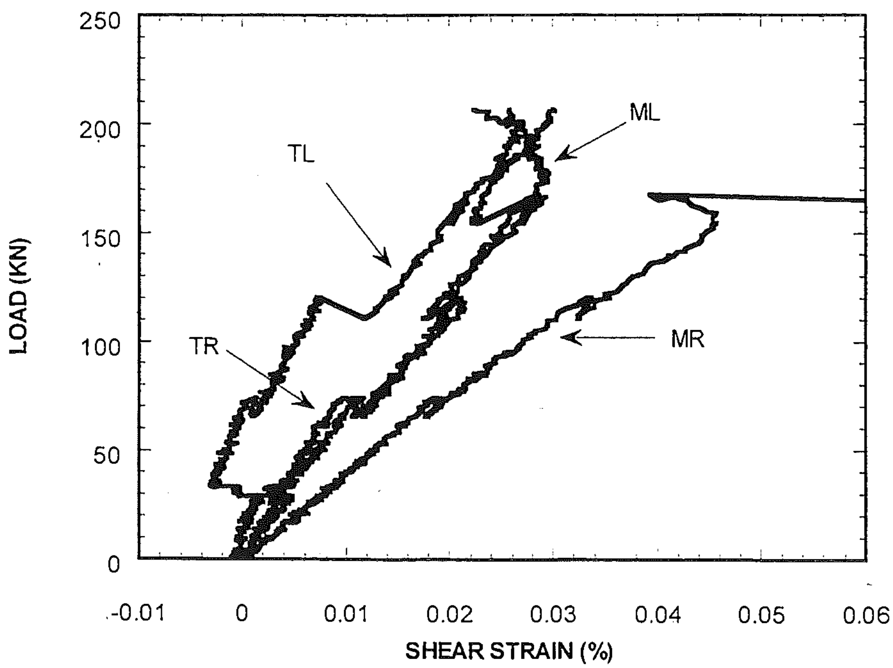


Figure 5.10 Load versus Shear Strain Curves from the Monotonic Shear Test

From Figure 5.10 it can be seen that neglecting these "jumps" the shear strains on the CFRP would have a linear relationship with the load. The shear strains are higher at the midsection than at the top of the CFRP wrap as is expected from the shear distribution of strains along the cross section of a RC beam.

The specimen subjected to fatigue cycles under shear load configuration failed by fatigue of three of the reinforcing bars in the extreme layer of reinforcement under the load point. A major vertical crack developed in the concrete at the midspan section including the CFRP sheet. Figures 5.11 and 5.12 show photograph of this test. Samples of the rebars that failed were analyzed under the microscope. Figure 5.13 and 5.14 show photographs of the surface of two coupons. A radial orientation of the grains of steel indicates a brittle failure. It was assumed that a small fatigue crack could initiate a brittle fracture of the section with low temperatures. Even if the type of failure of this specimen was different from the monotonic shear test, it is not considered representative of the overall behavior of a reinforced concrete beam using CFRP sheets as shear reinforcement. A beam with no stirrups is expected to behave in a brittle manner, especially when subjected to fatigue loads. In this particular configuration the longitudinal rebars were the only element able to perform load redistribution. If they fail, no load redistribution is possible and the beam will fail. Figure 5.15 shows the load-shear strain curves from the shear fatigue test. Similar to the monotonic test, shear strains are higher at the middle than at the top of the CFRP wrap. Except for the strain at the middle left, shear strains showed a linear relationship with the load.

5.3. Strengthening Level

Load versus deflection curves for the flexure tests were presented in Figure 4.1-4.2. A comparison is presented in Figure 5.16. As indicated previously for a better visualization of these graphs, the different fatigue curves were scattered along the deflection axis (i.e. origin was shifted on the x axis). From Figure 5.16 it is shown that under fatigue loads, the stiffness of the beam after 155500 fatigue cycles is higher than for the monotonic case. From the preliminary calculations, it was expected that the moment capacity of this section was 35.54 N-m and the CFRP will take a maximum stress of 745 MPa at failure (see last row of table 1A, Appendix a). From the experimental data, it was found that the maximum moment capacity of the beam was 50 N-m, a difference of 30% with the preliminary analysis.

As was predicted on the preliminary calculations, debonding of the laminate occurred before the full strength of the laminate could be reached. Figure 5.17 shows load-stress curves for the monotonic case, based on the strain values obtained from the strain gages (see Figure 5.3) and the CFRP modulus provided by the manufacturer. From this figure it can be observed that maximum stresses at mid span were 885 MPa, only 37% of the maximum strength capacity of the Sika laminate (2400 MPa). This experimental value (885 MPa) was 20% higher than the one predicted in Appendix A (734 MPa).

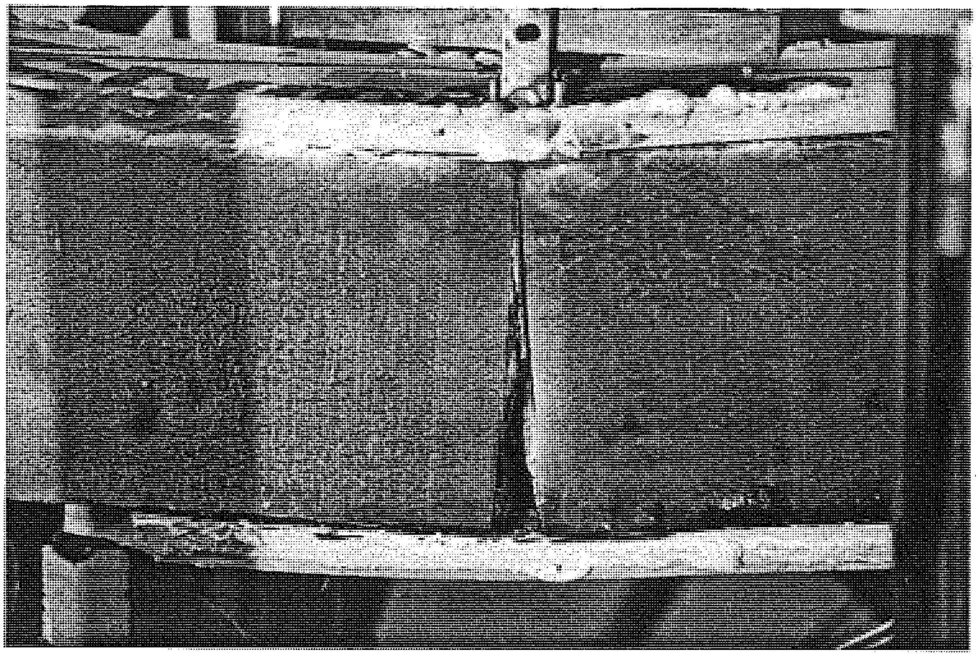


Figure 5.11 Fatigue Failure of the Shear Cyclic Specimen

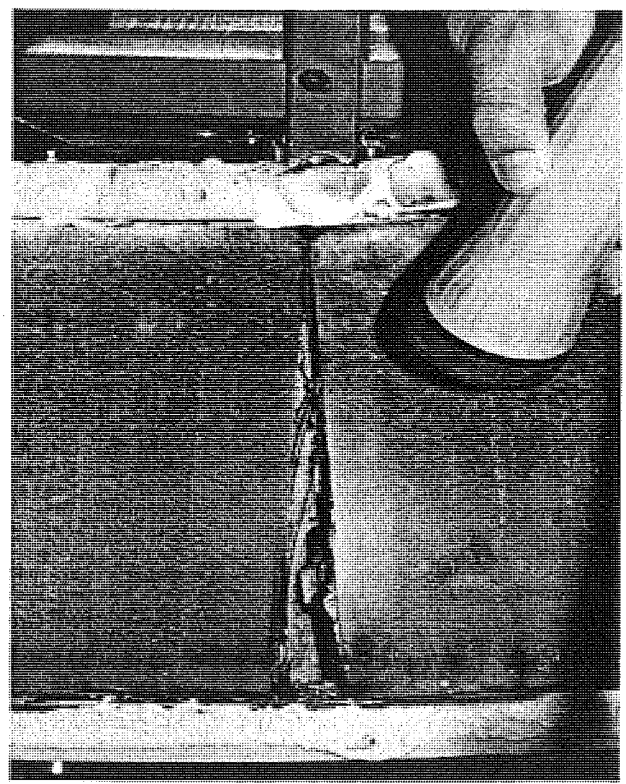


Figure 5.12 Detail of the Fatigue Failure of the Shear Cyclic Specimen

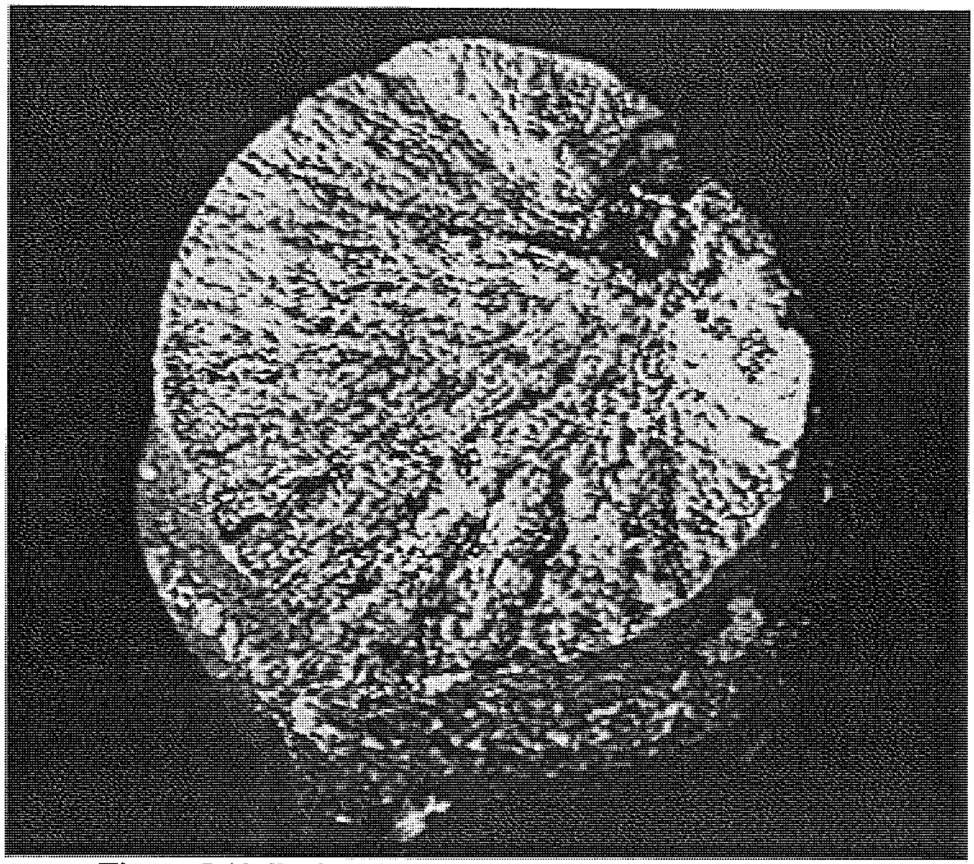


Figure 5.13 Fatigue Failure of a Steel Rebar (bar No.10)

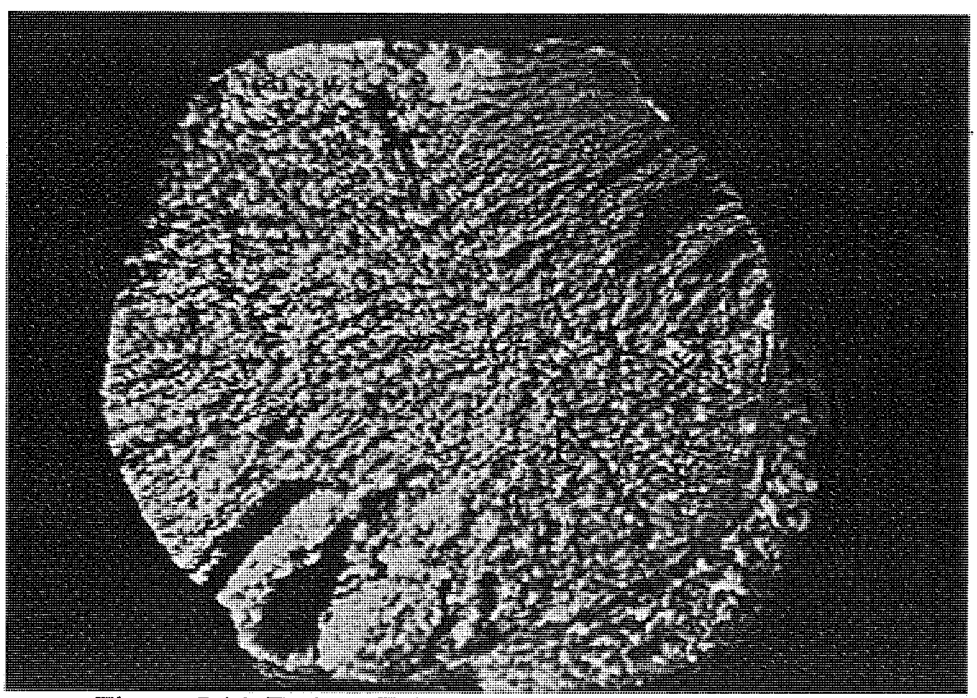


Figure 5.14 Fatigue Failure of a Steel Rebar (bar No. 13)

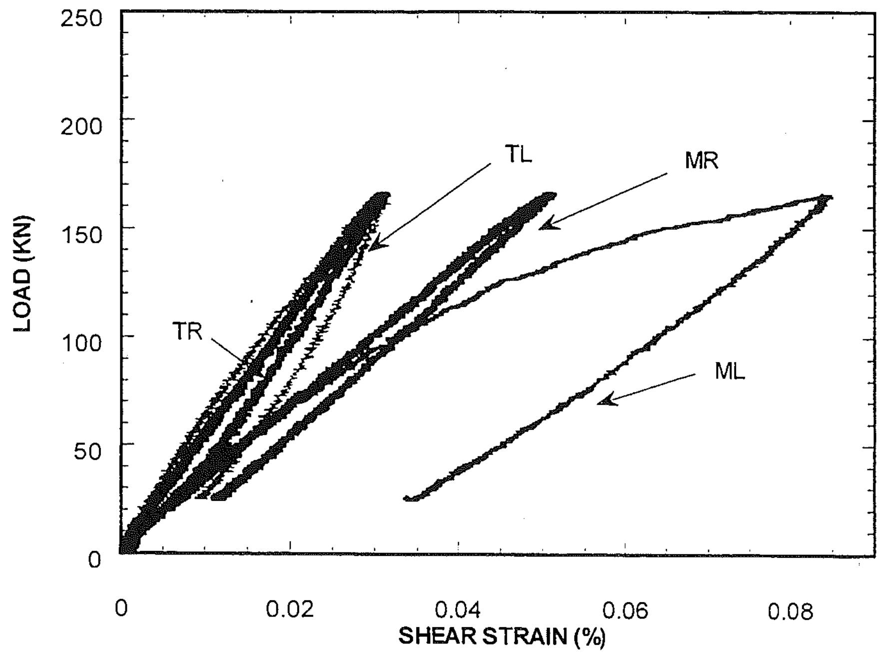


Figure 5.15 Load versus Shear Strain Curves from the Fatigue Shear Test

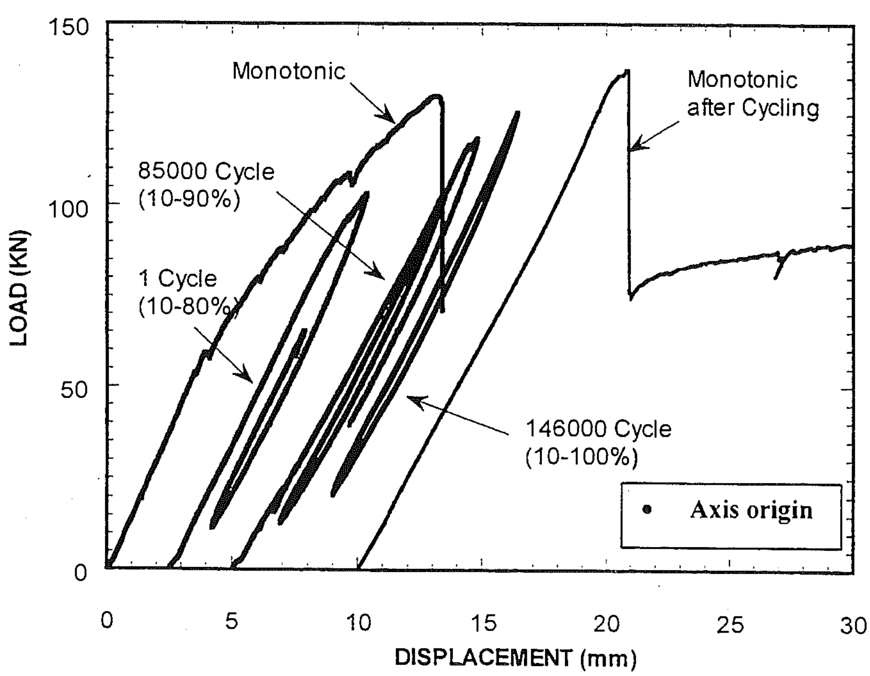


Figure 5.16 Load versusDeflection Curves for Monotonic and Fatigue Bending Tests

Using this experimental value of maximum strain, interfacial shear stress(τ_s) can be calculated as:

$$\tau_s = (\sigma_{CFRP} * t_f)/\ l_d,\ \sigma_{CFRP} = E_{CFRP}\ e_{CFRP}$$

where l_d is the length of the shear span and σ_{cfrp} is the average stress of the CFRP sheet at the midspan and t_f is the thickness of the CFRP plate.
For the monotonic case, the value of τ_s is equal to 1.39 MPa, a higher value than the one predicted in Appendix A (τ_s=1.15 MPa).

For the cyclic monotonic case, strain gages registered a maximum value of 554 MPa at the right end (R1), see Figure 5.18. The stress value at this point was only 23% of the maximum strength capacity. Strain gages placed at the midspan (M2) and the left side (L2) registered maximum strain values very close to the one registered by strain gage R1. This graph is an indication that a redistribution of stresses has occurred within the laminate probably due to the presence of some debonding along the interface, giving a more uniform pattern than the one registered on the monotonic case.

Since the maximum value of stress at the CFRP level was found at one of the ends, three different calculations for the value of the interfacial shear stress were carried out. The first calculation takes the average maximum stress value registered at the midspan times the area of the laminate and divides it by the total length of the shear span. This is a conservative calculation approximation since it is based on the assumption that the entire shear span will carry a uniform shear stress (as was calculated for the case of the monotonic bending specimen). The second calculation takes into account that the shear stress may not be uniform along the span and that partial debonding failure occurred at the end of the laminate and not at midspan. For the monotonic test, maximum stress value registered by L2 is multiplied by the area of the laminate and divided by the surface area from that point up to the end (l=521 mm). For the cyclic test, maximum stress value registered at R2 is multiplied by the area of the laminate and divided by the surface area from that point up to the end (l=521 mm). Finally, a strip of beam of the length equal to the distance between two strain gages located on the shear span (l=254 mm) is considered. The difference between the stresses in the laminate (L1-L2 and R1-R2) is used to calculate the value of τ (l=254 mm). Table 5.2 presents a summary of these calculations.

Table 5.2 **Interfacial Shear Values for the Bending Tests**

	τ, MPa (experimental, midspan)	τ, MPa experimental , ¼ span)	τ, MPa (experimental left span-right span)
Monotonic flexure	1.39	0.87	N.A. *1.62
Cyclic flexure	0.79	0.76	*1.58 1.05

*debonding failure occurred at this span
N.A. = value not available due to failure of strain gage L1.

It can be determined from the calculations presented above, all the τ values calculated from a difference between the values given by the strain gages located on the shear span that failed gave the higher value of the interfacial shear stress. Interfacial shear values calculated from the average stresses of the CFRP at the midspan represent a lower limit of this value. This table shows that the interfacial shear stress is not uniform along the shear span and that presence of high end plate stresses may influence the location of the debonding area.

For the shear tests, both monotonic and fatigue, the maximum capacity of the specimen was defined by the shear confinement that the CFRP sheets could confer to the beam. According to AASHTO, the shear contribution given by the concrete can be estimated as $V_c = 0.17\sqrt{f'c}\ bd$. For a f'c=23.24 MPa, this contribution is equal to 23 KN. Even though the strain readings revealed a low stage of strains on the surface of the CFRP wraps, indirect findings can be analyzed. Figure 5.10 shows the load-shear strain curves from the shear monotonic test. It can be seen that the first jump in all the curves is present at a load of 28 KN and correspond to the initiation of the first shear crack on the concrete. This value fits well with the theoretical concrete shear strength calculated above.

From the shear tests, shear stresses were calculated from the values of shear strain provided by the strain gages rosettes (see Figures 5.10 and 5.15) and a value of 4.40 GPa for the shear modulus found in the literature review [XX-48], see Table 5.3.
Al-Sulaimani et al. [XX-43] present an equation to evaluate the shear contribution of the CFRP wrap. It is assumed that peeling of the wrap will occurred when the maximum shear stress τ_{max} reaches the interface shear strength τ_{ult} (τ_{ult} =3.5 MPa). The shear distribution is assumed to be parabolic with maximum at the top and bottom of the strip. Value of τ_{ave} (average) can be estimated from the experimental results:

$$Vp = 2(\tau_{ave}\,(d^*hw)/2)$$

where Vp is the shear contribution of the CFRP wrap (V=Vp+Vc), d= distance from extreme compression fiber to centroid of steel reinforcement and hw = depth of the strip.
Assuming Vc=23 kN, values for Vp and τ_{ave} were calculated. Experimental shear stresses as well as values for τ_{ave} are presented in Table 5.3.

Table 5.3 Shear Contribution of the CFRP sheet

Source	Parameter	Monotonic Shear Test	Fatigue Shear Test
Experimental	CFRP Shear contribution, N	Vp = 80 KN	Vp=60 KN (at the moment of failure)
[XX-43]	Interfacial shear stress, τ_{ave}, MPa	2.77	2.08
Calculated*	τ = G x shear strain, MPa	1.89(max)0.92(min)	3.70(max)1.32(min)

* Shear strains were obtained from experimental results, shear modulus G was found in the literature review.

From the monotonic test, it can be seen that the Tonen CFRP sheet wrapped around three sides of the beam gave an additional shear capacity of 80 KN. A maximum shear strain (0.045%) was obtained for the strain gage rosette located on the middle right side. The shear stress at this level is equal to 1.89 MPa. Following the equation given by [XX-43] the bond strength of the interface between the CFRP laminate and the concrete was found to be 2.77 MPa which is 3.4 times the shear strength of the concrete ($0.17\sqrt{f'c}$). Confidence of the data collected by the strain gages rossettes is being doubted due to the presence of bond failure of the strain gages.

For the cyclic test, the bond between the strain gage and the CFRP surface also seems to be deteriorated even more notoriously by the combined effect of the cold temperature and fatigue. Figure 5.15 shows selected load-strain curves for the last fatigue cycle. Two strain gages failed early on the test. However, from the data collected, it was found that a maximum shear strain (0.084%) was obtained for the strain gage rosette located on the middle left side. The shear stress at this level is equal to 3.70 MPa. Following the equation given by [XX-43] the bond strength of the interface between the CFRP laminate and the concrete was found to be 2.08 MPa which is 2.5 times the shear strength of the concrete ($0.17\sqrt{f'c}$).

5.4. Temperature and Fatigue Effect

In this part of the study, no control beam was tested under room temperature; thus it is not possible to make a quantitative assessment of the effect of cold temperature on the overall behavior of the specimens. However, a qualitative evaluation can be made in comparison to previous bending and shear tests carried out at room temperature.

From the previous bending tests, all the beams tested had a longitudinal reinforcement ratio smaller than the beam tested at low temperature. For a beam with a longitudinal reinforcement of 0.54 ρmax and 1 strip of Sika Carbodur of 100 mm (beam No.8-1), the failure load was 33% higher than the control beam. The Sika plate had a maximum tensile stress of approximately 600 MPa (25% of the maximum strength capacity of the laminate). The bending beam tested monotonically under low temperatures had a failure load 120% higher than the theoretical moment capacity of the beam without the CFRP reinforcement. The Sika plate had a maximum tensile stress at midspan of 885 MPa (37% of the maximum strength capacity of the laminate). It can be concluded that the low temperatures did not affect the flexure strengthening level provided by the Sika system.

For the bending tests at low temperatures, the same failure mode was observed also for beams tested at room temperature. Debonding of the laminate due to shear failure of the interface between the CFRP and the concrete led to a premature failure of the beam since it did not allow the CFRP to reach its maximum strength.

It can be concluded that no evident effect of the cold temperature is observed for the Sika strengthening system.

From the previous shear tests, a beam with a longitudinal reinforcement of 1.41 ρmax, no stirrups and wrapped with 1 layer of the Tonen sheet (beam No. 2) obtained a failure load 150% higher than the control beam. The shear beam tested monotonically at low temperature (0.89 ρmax, no stirrups and wrapped with 1 layer of the Tonen sheet) had a failure load 800% higher than the predicted shear concrete strength. It can then be conclude that the low temperatures did not affect the shear strengthening level provided by the Tonen system.

For the shear beam tested monotonically at low temperature, failure was similar to that observed in the previous shear tests at room temperature. Propagation of a major concrete shear crack from the load point to one of the supports (left side) led to debonding of the CFRP sheet along this crack due to the relative displacement of the two concrete blocks. The shear specimen subjected to cyclic fatigue loads failed by brittle failure of three of the reinforcing bars in the extreme layer of reinforcement. It was considered that a combination of low temperature and fatigue could have contributed to this type of failure. However, since this failure was by the steel rebars, one can also conclude that there was no evidence that cold temperature had an affect on the Tonen strengthening system.

Other effects of fatigue loading can be observed by comparison with specimens tested under monotonic load.

Comparing load-strain curves for the monotonic and last cycle of the fatigue bending test, it can be observed that a redistribution of stresses and strain has occurred along the CFRP plate. A higher number of flexural cracks were observed under fatigue. Due to the fact that the beam was insulated all the time, it was not possible to observe deterioration of the interfacial layer. Calculated values of the interfacial shear stress were always lower for the case of fatigue test than for the monotonic test with a minimum and maximum difference of 3-40%. It was concluded that fatigue cycles at low temperatures decrease the strength of the interface between the CFRP laminate and the concrete.

For the shear tests, because fatigue at low temperature led to failure of rebars, there was no evidence of fatigue on the CFRP sheet. Shear strains registered on the surface of the CFRP at the top height were at the same level as those for the monotonic case indicating no deterioration of bonding at this level. At the mid height of the CFRP sheet, an increase in shear strain was recorded with an increasing number of fatigue cycles. It was expected that deterioration of the bond between the CFRP wrap and the concrete surface at this level, probably related to the fatigue failure of the steel rebars allowed a larger deformation of the CFRP sheet under fatigue load.

Finally it was concluded that the interface between the CFRP laminate and the concrete is the weakest link of this strengthening system and is likely to first register effects of low temperature and fatigue load.

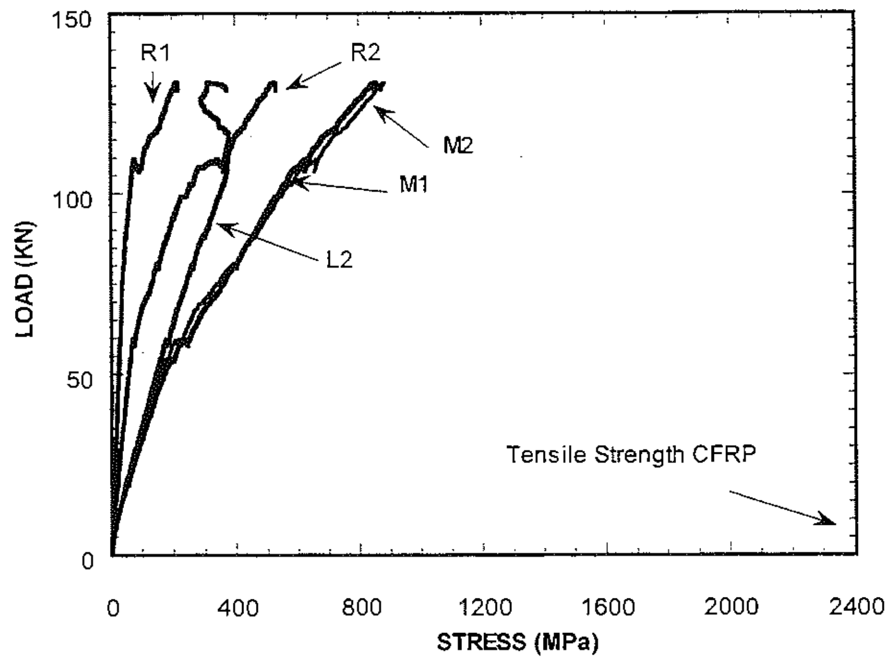


Figure 5.17 Load versus Stress Curves for Monotonic Flexure Test

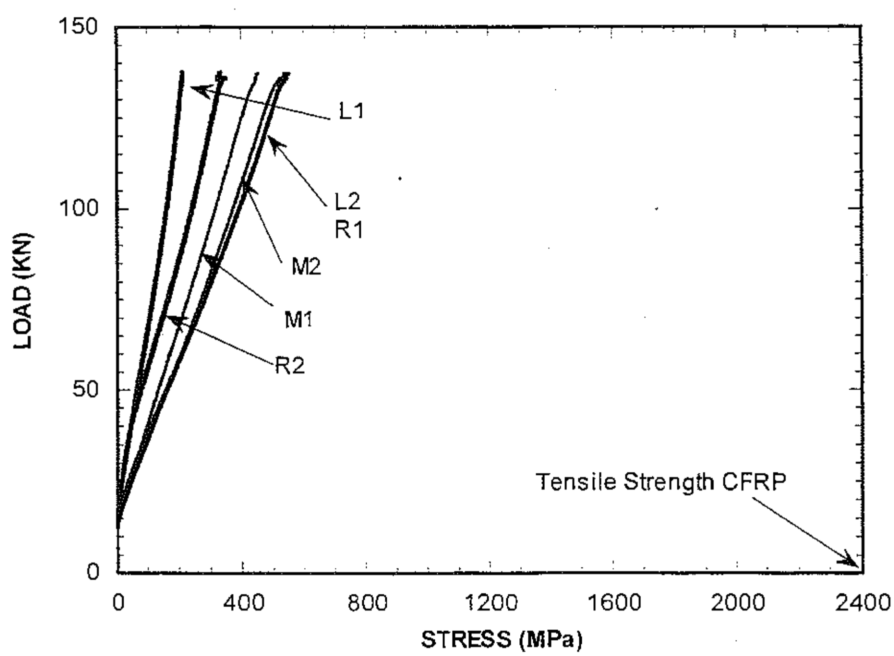


Figure 5.18 Load versus Stress Curves for Monotonic after Cycling Bending Test

6. CONCLUSIONS

1. Prior tests (Task 2) showed that the failure mode of RC beams strengthened with CFRP Sika plates and loaded in monotonic bending at normal room temperature was by delamination of the CFRP plate. The limited tests carried out in this task with low temperature (-29 °C) and cyclic fatigue loading, suggest that the failure mode remains the same, that is by delamination.

2. The strain data of the cyclic fatigue test in bending showed redistribution of strains (thus stresses) in the CFRP plate with an increasing number of cycles. A more uniform strain pattern was achieved suggesting that slow delamination of the plate occurred during cycling. Higher strains at the end of the plate confirmed the extension of delamination toward that section and subsequent delamination failure at that section.

3. Values of the interfacial shear stress from the strains recorded by the gages showed that the interfacial strength at failure (1.62-1.58 MPa) was similar for both the monotonically flexure tested beam and the fatigue flexure beam after 155500 cycles

4. Failure in the shear beam subjected to monotonic loading at -29 °C occurred by shear delamination of the CFRP Tonen sheet followed by shear failure of the concrete. The shear delamination was due to the propagation of a diagonal shear crack within the concrete that extended from the load point to the support.

5. Failure in the shear beam that was subjected to cyclic fatigue loading at -29 °C was initiated by failure of one of the reinforcing bars in the first layer of steel, and shortly followed by failure of two additional bars. Subsequent analysis suggested that failure of the rebars was by brittle fracture due to the low temperatures at which the tests were preformed.

6. Increase in the shear strain obtained with cyclic shear loading from the rosette gage placed at midspan along the vertical axis of the beam suggest that some delamination and cracking were occurring at that section. It is likely that a shear failure would have occurred in a manner similar to the monotonically tested beam, should failure of the reinforcing bar not have occurred.

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APPENDIX A

Design of Reinforced Concrete Beams with CFRP for Low Temperature Test

Design of the reinforced concrete beams strengthened with the CFRP sheets was defined prior to the experimental tests. Expected maximum load was necessary to select the load cell to be used for the experimental load set-up. It was also desired to obtain the highest strengthening level that the CFRP plate could provide for a given steel reinforcement level.

Calculation of reinforcement ratio for reinforced concrete beam with no CFRP

$$\rho_b = \frac{0.85\beta_1 f'c}{f_y} \times \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y}\right) = 0.00249\text{, f'c=24.13 MPa}$$

$\rho_{max} = 0.75\rho_b$

As= 3#13+1#10 = 458 mm^2,

$\rho = 0.0159 = 0.85\rho_{max}$

Moment Capacity = 22.64 kN-m

Moment Capacity for RC beam with CFRP plate

A spreadsheet provided by Sika Corporation was used to design the RC beam strengthened with a layer of CFRP plate. According to Sika Co., this spreadsheet is based on experience of European manufacturers and has been calibrated with ACI fundamentals. The design methodology follows the strain compatibility method where the concrete stress-strain curve is built according to the Swiss code (SIA162, 1989)[SCD-7]

For a particular RC beam (geometry, steel reinforcement ratio, steel yield strength, concrete compressive strength already defined), the spreadsheet can provide moment value, concrete strain, steel strain and CFRP strain for a particular percentage of the ultimate moment capacity. Delamination of the CFRP plate is also checked according to a procedure developed by the Swiss Federal Laboratories for Material Testing and Research (EMPA). A copy of the print-out of the spreadsheet is provided at the end of this section.

For the defined bending beam, different widths of Sika plate were input into the spreadsheet. Moment capacity was defined as the moment when the top compressive concrete strain is equal to 0.003. It was also checked that steel had yielded at this level. Strain at the level of the CFRP plate was calculated as well as the expected development length required by this methodology.

Following the uniform shear model developed in the freeze-thaw report (report from task No.4) appendix A, shear stresses at the concrete-CFRP interface were calculated. Also, assuming a interfacial shear strength of $\tau_1=0.17\sqrt{f'c}$ = .813 MPa and $\tau_2=0.34\sqrt{f'c}$ = 1.63 Mpa (assumed as feasible range of values for the shear strength of the interface based on previous bending tests) , development length is checked for every case.

Since the shear forces are uniform in the shear span of a four point bending test, a uniform interfacial shear stress (τ_s) model is considered.

Therefore $\quad l_d = T_{CFRP} /(b^*\tau_s) = (\sigma_{CFRP} * t_f)/ \tau_s \quad \sigma_{CFRP} = E_{CFRP}\, e_{CFRP}$

$\tau_s = (\sigma_{CFRP} * t_f)/ l_d$

where l_d is the development length and T_{CFRP},σ_{cfrp} are the tensile force and stress of the CFRP sheet at the point under one of the concentrated loads, b and t_f are the width and thickness of the CFRP plate and τ_s is the shear strength of the interface. To calculate τ_s, l_d was assumed as the shear span for the CFRP laminate and σ_{CFRP} is calculated from the strain of the CFRP for every case. To calculate l_d , τ_s is set equal to τ_1 and τ_2, and σ_{CFRP} is calculated from the strain of the CFRP for every case.

Summary of the results of this design is presented in Table 1A. A Sika plate of a width equal to 102 mm is chosen for a shear span length of 762 mm. A wider CFRP plate will lead to compressive failure on the top concrete layer. A narrower CFRP plate will fail by delamination giving a lower moment capacity. This final selection is checked following the strain compatibility design methodology presented in the freeze-thaw report. It is found that this selection provide an interfacial shear stress of 1.15 MPa, which is between the range giving by τ_1 and τ_2, data. Results of this calculation are also presented in Table 1A.

Table 1A. Summary of Design Calculations

Sika plate Width (mm)	Moment (kN-m)	Yield of steel reinf.	Strain of CFRP plate	Interf. Shear stress (MPa)	Shear span (mm)	Development length (mm) for the Sika plate		
						Software	(τ_1)	(τ_2)
25.4	31.18	Yes	0.00759	1.84	762	1727	1727	864
51.0	35.25	Yes	0.0065	1.58	762	1270	1473	737
76.0	39.32	Yes	0.00542	1.31	762	889	1219	610
102	40.67	Yes	0.00433	1.05	762	559	991	508
127	43.38	Yes	0.0043	1.05	762	559	991	508
152	42.03	No	0.00325	0.79	762	508	991	508
178	44.74	No	0.00325	0.79	762	508	762	381
203	48.80	No	0.00325	0.79	762	508	762	381
*102	35.54	Yes	0.00474	1.15	762	N.A.	N.A.	N.A.

*revised final design values

N.A. not applicable

Design of stirrups

Stirrups (#10 close stirrup) were provided for the shear force at the ultimate moment capacity calculated previously. AASTHO provisions were considered for the amount and spacing of the stirrups. Figure 1A shows the stirrup layout for this specimen.

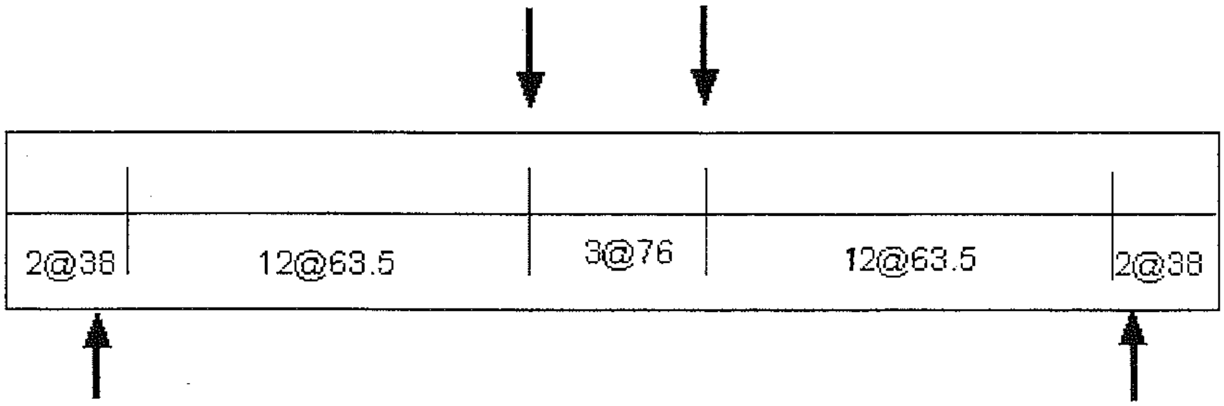

Figure 1A. Stirrup Layout for the Bending Test

Shear Capacity for RC beams wrapped with CFRP sheets

Different researchers have evaluated in a different way the contribution of the CFRP wraps to shear reinforcement. Since the peeling-off effect is present on both tests fatigue and cyclic, it is therefore necessary to evaluate this effect in the calculation of the shear confinement. [XX-43] presents an equation to evaluate the shear contribution of the CFRP wrap. It is assumed that peeling of the wrap will occurred when the maximum shear stress τ_{max} reaches the interface shear strength τ_{ult} (τ_{ult} =3.5 MPa). The shear distribution is assumed to be parabolic with maximum at the top and bottom of the strip. Vp is the shear contribution of the CFRP wrap defined by:

$$Vp = 2(\tau_{ave}\,(d*hw)/2)$$

where τ_{ave} is the average value of the shear strength over the height of the wrap.

Assuming $V_c = 0.17\sqrt{f'c}\,bd$, V_c=23 kN (for f'c = 24.13 MPa)
Assuming $\tau_{ave} = \tau_{ult}$ =3.5 MPa Vp = 100 kN

The total shear capacity of the section (V) is calculated as: V= Vc+Vp = 123kN

print-out of the design spreadsheet provided by SIKA

MAIN TABLE

Percent of Ultimate L-stress	Mn (k-ft)	Flexure Check ($Mn<\phi Mn_{ult}$?)	Flexure Check ($Mn>1.4M_D+1.7M_L$?)	Concrete crushing check	Has steel yielded?	Steel failure check	Delamin. check	Development length (in)	CFRP length check	Check for T section	OVERALL CHECK
0	17	OK*	OK	OK	YES	OK	N/A	N/A	N/A	N/A	OK
10	18	OK	OK	OK	NO	OK	OK	20	OK	N/A	NG
20	23	OK	OK	OK	NO	OK	OK	20	OK	N/A	NG
30	27	OK	OK	OK	NO	OK	OK	20	OK	N/A	NG
40	30	OK	OK	OK	YES	OK	OK	22	OK	N/A	OK
50	33	NG	OK	NG	YES	OK	OK	35	NG	N/A	NG
60	36	NG	OK	NG	YES	OK	OK	50	NG	N/A	NG
70	39	NG	OK	NG	YES	OK	OK	68	NG	N/A	NG
80	41	NG	OK	NG	YES	OK	OK	89	NG	N/A	NG
90	44	NG	OK	NG	YES	OK	OK	112	NG	N/A	NG
100	46	NG	OK	NG	YES	OK	OK	139	NG	N/A	NG

* For the unstrengthened section, the check is: $\phi Vn>1.4VD+1.7VL$?, $\phi Mn>1.2(MD+ML)$? and pact < 0.75 pbal?

UNSTRENGTHENED BEAM FLEXURAL CHECK

ϕMn= 15 ft-k
ρ_{ACT}= 0.01596
ρ_{bal}= 0.02568

$\phi Mn>1.2(Md+ML)$? OK
$\rho_{act}<0.75\ \rho_{bal}$? OK
OVERALL OK

USER INPUT

New loading conditions:
M_D= 1 klf
M_L= 1 klf
V_D= 1 k
V_L= 1 k
$1.2M_L+1.2M_D$= 2 k-ft
$1.7M_L+1.4M_D$= 3 k-ft
$1.4V_D+1.7V_L$= 3.1 k
$L_{AVAILABLE}$= 3 ft
Span= 6.0 ft
Include initial conditions (Y/N)? Y

k-ft

Existing Beam Properties:
As = 0.71 in^2
fy= 60 ksi
Es= 29000 ksi
fc= 3.5 ksi
h = 8.00 in.
d = 5.56 in
b = 8 in
β_1= 0.875
Tee-flange depth= 8.0 in *
Tee-web width= 8.0 in *
Ts = 42.6 k

CarboDur S CFRP Properties:
# of strips: 1
b_L= 4.00 in
t_L= 0.047 in
f_{LU}= 348 ksi
E_L= 22481 ksi
R_L= 0.7
te= 0.481197 in.
* If not a Tee, input the same values as for h and b, respectively

Development Length and Delamination Factors:
τ_{CA} = 0.319 ksi
ξ = 0.00606
χ = 0.00105
ψ_R= 0.6
τ_K = 1.88 ksi
K = 4.35 ksi
Strip spacing= 8 in
A_{LREAL}= 0.18800 in^2

ANALYSIS OF THE INITIAL CONDITIONS (ASSUMED ELASTIC)

M_D= 1 ft-k
c (elast) = 2.19 in
f_s = 3.50 ksi
f_c = 0.28 ksi
ε_{d}= 0.00012
ε_{d}= 0.00008

* If moments cannot be expressed like this, input the factored moments directly.

CALCULATED STRAINS, LOCATION OF THE NEUTRAL AXIS, SHEAR FORCE AT DELAMINATION, ETC.

% of ult. L-stress	T_L (k)	ε_L	ε_c	ε_s	$\phi_R Va$ (k)
0	0.0	0.00000	0.00300	0.00615	N/A
10	6.5	0.00108	0.00111	0.00056	26
20	13.1	0.00217	0.00164	0.00116	25
30	19.6	0.00325	0.00222	0.00173	24
40	26.2	0.00434	0.00294	0.00226	22
50	32.7	0.00542	0.00381	0.00275	21
60	39.3	0.00650	0.00486	0.00318	21
70	45.8	0.00759	0.00611	0.00355	21
80	52.4	0.00867	0.00760	0.00385	21
90	58.9	0.00976	0.00937	0.00407	21
100	65.4	0.01084	0.01146	0.00418	22

CALCULATION OF NEUTRAL AXIS LOCATION AND CONCRETE STRAINS BASED ON TWO MODELS; CHOSING THE APPROPRIATE MODEL.

% of ult. L-stress	c (ε_c<.002)	Equilibrium equation	c (ε_c>=.002)	Equilibrium equation	ε_c (model 1)	ε_c (model 2)	c used (in)	k_2 used
0	N/A	N/A	N/A	N/A	N/A	N/A	2.05	N/A
10	3.89	0.000	4.03	0.000	0.00111	0.00118	3.89	0.35
20	3.34	0.000	3.35	0.000	0.00164	0.00164	3.34	0.36
30	3.18	0.000	3.18	0.000	0.00222	0.00222	3.18	0.38
40	3.28	0.000	3.18	0.000	0.00309	0.00294	3.18	0.40
50	3.21	-11.688	3.26	0.000	not solved	0.00381	3.26	0.42
60	2.88	-25.030	3.39	0.000	not solved	0.00486	3.39	0.44
70	2.62	-37.044	3.54	0.000	not solved	0.00611	3.54	0.45
80	2.40	-48.089	3.72	0.000	not solved	0.00760	3.72	0.46
90	2.21	-58.403	3.90	0.000	not solved	0.00937	3.90	0.47
100	2.06	-68.151	4.10	0.000	not solved	0.01146	4.10	0.47

VERSION 1.0

UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 6: Behavior of Beams Subjected to Freeze-Thaw Cycles

by

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16. Abstract

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:
Volume 1 – Summary Report
Volume 2 – Literature Review
Volume 3 – Behavior of Beams Strengthened for Bending
Volume 4 – Behavior of Beams Strengthened for Shear
Volume 5 – Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications

The part of the investigation dealing with the flexural testing of reinforced concrete beams with glued-on CFRP plates subjected to different numbers of freeze-thaw cycles is the subject of this volume (volume 6). Results are also analyzed, compared, and discussed. The experimental program comprised forty-eight reinforced concrete beams. The specimens were subjected to up to 300 freeze-thaw cycles according to ASTM C666. For every parameter, three beams were tested in bending at 0, 100, 200 and 300 cycles. Parameters investigated were two different adhesive systems, the Tonen CFRP sheet system (MBrace), and the Sika CFRP system (Carbodur); and a cracking stage where a precracked condition simulates cracking conditions in the field prior to strengthening. Control specimens (RC beams with no CFRP laminate externally glued-on) were also subjected to 0, 100, 200 and 300 cycles freeze-thaw cycles. Conclusions are drawn and some recommendations for design are suggested.

17. Keywords bond, carbon fiber, composites materials, laminates, rehabilitation, reinforced concrete, repair, strengthening systems	18. Distribution Statement No restrictions. This document is available to the public through the Michigan Department of Transportation		
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The opinions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors.

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PREFACE

This project titled: *"Repair and Strengthening of Reinforced Concrete Beams using CFRP Laminates"* is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of the project were:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) glued-on plates for repair and strengthening of concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

The project consisted of 8 tasks as follows:

- A report containing a literature review and a comprehensive synthesis of the latest state of knowledge on the glued -on FRP technique (Task 1);
- Laboratory testing and verification of the selected CFRP glued-on technique according to the proposed experimental program: bending (Task 2), shear (Task 3), freeze-thaw (Task 4), temperature and high cyclic amplitude load (Task 5);
- An interim and final report summarizing the experimental results (Task 6). The interim report will cover the bending and freeze-thaw tests;
- A summary of field specifications and "how to" details for implementation in field applications;
- Guidelines for design based on the experience developed from the experimental work (Task 7);
- Field monitoring of application of the technique to one bridge selected by MDOT (Task 8a);
- Bridge testing before and after application of the glued-on plate (Task 8b to be conducted by professor A. Nowak, U of M)

This report summarizes the experimental program on freeze-thaw tests as per Task 4.

ABSTRACT

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report is part of a larger investigation on the use of carbon fiber reinforced plastic (CFRP) sheets for repair and strengthening of reinforced and prestressed concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, 2) to synthesize existing knowledge, 3) to identify optimum parameters for successful implementation, 4) to develop procedures for implementation in the field, and 5) to adapt the technique to the specific conditions encountered in the state of Michigan.

The experimental program includes four main parts: 1) tests of RC beams strengthened in bending; 2) tests of RC beams strengthened in shear; 3) freeze-thaw tests of strengthened beams followed by their test in bending; and 4) tests in bending and shear of strengthened beams under low temperature (-29° C) and high amplitude cyclic loading.

The part of the investigation dealing with the flexural testing of reinforced concrete beams with glued-on CFRP plates subjected to different numbers of freeze-thaw cycles is the subject of this report. Results are also analyzed, compared, and discussed. The experimental program comprised forty-eight reinforced concrete beams. The specimens were subjected to up to 300 freeze-thaw cycles according to ASTM C666. For every parameter, three beams were tested in bending at 0, 100, 200 and 300 cycles. Parameters investigated were two different adhesive systems, the Tonen CFRP sheet system (MBrace), and the Sika CFRP system (Carbodur); and a cracking stage where a precracked condition simulates cracking conditions in the field prior to strengthening. Control specimens (RC beams with no CFRP laminate externally glued-on) were also subjected to 0, 100, 200 and 300 cycles freeze-thaw cycles. Conclusions are drawn and some recommendations for design are suggested.

The experience gained during this project should contribute to a better understanding of the behavior of these new strengthening systems under different environmental conditions.

EXECUTIVE SUMMARY

This report presents the summary of experimental work, laboratory testing, and analysis of results for task 4 of the current research project which deals with flexural testing of reinforced concrete beams with glued-on CFRP plates subjected to an increasing number of freeze-thaw cycles. Results are also analyzed, compared, and discussed.

Based on the results from the experimental work, the following conclusions were drawn:

1. For the control reinforced concrete specimens no decrease in the moment capacity or shear strength due to the freeze-thaw cycles was observed. However, a decrease in the maximum deflection was observed.

2. For specimens strengthened with CFRP sheets an overall decrease in the moment capacity as well as the maximum deflection was observed with an increase in number of freeze-thaw (F.T.) cycles. Precracked beams using the Tonen system presented the higher rate of decrease of moment capacity (38% average for 300 F.T. cycles). Non-precracked beams also strengthened with the Tonen system led to an average decrease of 20% for 300 F.T. cycles.

3. The maximum moment capacity of beams strengthened with the Sika system decreased 13% on average for 200 F.T. cycles and 4% average for 300 F.T. cycles. This variation was attributed to the influence of different type of failure modes.

4. The average deflection at maximum load was very sensitive to the effect of the freeze-thaw cycles and the cracking condition. For the Tonen precracked beams a reduction of 43% in deflection was found after 300 F.T. cycles whereas for the Tonen non-precracked beams the decrease was of 19%. Beams using the Sika system showed a smaller rate of decrease in deflection at maximum load (15% average for 200 F.T. cycles and 7% for 300 F.T. cycles)

5. With the Tonen system, the values of normalized shear stress (v_n) for the same number of freeze-thaw cycles were higher for the flexure-delamination failure than for the shear-delamination failure. It was concluded that shear cracks accelerate the interfacial crack propagation.

6. With the Tonen system, precracking the beam influences the decrease in the average normalized shear stress with the freeze-thaw cycles. For 300 freeze-thaw cycles, precracked beams had a decrease of 39% (compared with the strength at zero F.T. cycles) whereas the decrease for the non precracked beams was 22%. However the normalized shear stress after 300 F.T. cycles remained almost the same: $v=0.20\sqrt{f'c}$ for

precracked beams, and v=0.19$\sqrt{f'c}$ for non-precracked beams. It can be shown that at zero F.T. cycles the cracking condition influences the capacity of the beam, whereas after 300 F.T. cycles, the freezing and thawing effect dominates.

7. For the Sika system, ignoring vertical shear failure, the decrease in the normalized average shear stress at failure load due to the effect of the freeze-thaw cycles seemed to be less significant than for the Tonen system. A decrease of 10% was observed for 300 freeze-thaw cycles, leading to a value of 0.36$\sqrt{f'c}$.

8. For both strengthening systems (Tonen and Sika), the delamination length found in specimens that failed either by shear-delamination or flexure-delamination was located between the end of the CFRP laminate and the maximum bending moment region.

9. The delamination length was quite uniform for both strengthening systems. For the Tonen system, values of delamination length varied between 140 to 180 mm. For the Sika system, the range was even more narrow, 220-250 mm. No influence of either the number of freeze-thaw cycles or the type of failure mode was observed on delamination length.

Some reccomendations for design and retrofit use based on this study are summarized as follows:

1. Freeze-Thaw (F.T.) cycles influence the behavior of reinforced concrete beams with glued-on Carbon Fiber Reinforced Plastic (CFRP) laminates. According to this study, with the Tonen system a maximum decrease of 29% of flexural capacity could be expected after 200 FT and 38% after 300 F.T. cycles. With the Sika system, the maximum decrease observed was of 13% after 200 F.T. cycles. It should be pointed out that the influence of the Freeze-Thaw cycles may also affects the concrete strength, but its effect cannot be easily observed from testing a reinforced concrete beam in bending; it is possible that this dual effect explains the decrease in strength. Unless some additional tests are carried out, as a first design approximation, it is recommended that a reduction of 40% in horizontal shear strength be taken to account for freeze-thaw exposure.

2. The value of the horizontal interfacial shear strength can be taken conservatively as 0.17 $\sqrt{f'c}$ for both strengthening systems. Preliminary analyses indicate that this value may be close to 1.70 MPa for Tonen system and 2.64 MPa for the Sika system, considering the effect of the different strengthening level provided by the two systems. Further study of the interface bond behavior is needed in order to refine this value.

3. The minimum value of the development length (or anchorage) of the CFRP laminate should be based on the value of 0.17 $\sqrt{f'c}$. This value could be modified by results from further investigations. It is recommended that the bonded length of the CFRP should

be as long as possible in order to avoid an interfacial bond failure and to have a more efficient use of the CFRP sheet strength.

4. Since delamination seems to be controlled by the interface bond between the CFRP laminate and the concrete, which is also controlled by the concrete strength, it is strongly recommended to insure very good surface preparation before application of the strengthening system.

1. INTRODUCTION

This study is part of a research project at the University of Michigan supported by the Michigan Department of Transportation and the Great Lakes Center for Truck and Transit Research. The project title is "Repair and Strengthening of Reinforced and Prestressed Concrete Beams Using Carbon Fiber Reinforced Plastic (CFRP) Glued-on Plates". The study is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of this project are to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, to synthesize existing knowledge and develop procedures for implementation in the field, and to adapt this technique to the specific conditions encountered in the State of Michigan. The project consists of 8 tasks. Task 4 deals with the flexural testing of reinforced concrete beams with glued-on CFRP plates subjected to different numbers of freeze-thaw cycles. It is the subject of this report.

1.1 Organization of this Report

This report presents the summary of experimental work, laboratory testing, and analysis of the results for task 4 of the current research project: Flexural test of reinforced concrete beams with glued-on CFRP plates subjected to an increasing number of freeze-thaw cycles.

Chapter 2 summarizes known prior research on CFRP laminates subjected to freeze-thaw cycles. It describes how the proposed experimental project studies some of the unknowns about the performance of this strengthening technique.

Chapter 3 presents the experimental program. The main experimental parameters, the material properties, fabrication of specimens, and test set-up and instrumentation are specified.

Chapter 4 presents the test results from the bending tests of specimens subjected to the different freeze-thaw cycles.

Chapter 5 presents the interpretation and analysis of the experimental results.

Chapter 6 presents the conclusions based on this experimental study.

Chapter 7 summarizes the recommendations for design and retrofit use based on this study.

Chapter 8 provides an extensive list of references.

Chapters 9, 10 ,11 and 12 present appendixes A, B C and D.

2. LITERATURE REVIEW

The rehabilitation technique of external reinforcement of concrete elements by the use of CFRP sheets can not be effectively utilized in cold regions without investigating the durability of this new material in low temperature environments.

In the following sections the results of a literature review, aimed at investigating freeze-thaw durability of concrete elements strengthened by CFRP sheets are summarized. Table 2.1. highlights the most important parameters found in this literature review.

2.1.1. DE - University of Delaware, Delaware Department of Transportation

Reference: [DE-2]

At the University of Delaware, tests were performed to investigate the environmental durability of composite fiber materials: Aramid, E-glass and graphite (carbon) fibers. The environmental durability studies involved subjecting the 60 small scale size beam specimens to aggressive environments (the beams were exposed to repeated wet/dry and freeze/thaw cycles while submerged in a solution of calcium chloride.) Exposure times were varied and the flexural capacity of the exposed beam was compared with that of control beams. Once the beams completed the environmental cycling they were loaded to failure in four point bending. The results of the test indicate that the beams reinforced with the graphite fabric have greater environmental durability than those with Aramid or E-glass. After being subjected to environmental conditions, only graphite reinforced beams maintained nearly all of their strength advantage over the unwrapped beams. While both the Aramid and E-glass reinforced beams showed a 36 % decrease in strength after 100 wet/dry cycles, the graphite reinforced beams dropped in strength by only 19 % (as compared to a 12 % drop in strength of the unstrengthened beams). For the freeze-thaw test, a decrease of 21% on the strength of the graphite reinforced beams was reported after 100 freeze-thaw (F.T.) cycles (compare with 17% of unwrapped.) Of the two conditions showed above, the wet/dry cycling led to greater degradation. The tests also revealed that the environmental exposure could lead to a changed mode of failure compared to non-exposed specimens. Partial debonding was experienced prior to failure.

2.1.2. CAN - Canadian Institutes and Universities

Reference: [CAN-2]

Hoa et al. conducted an experimental program where beams of dimensions 51 x 76 x 279 mm were externally reinforced with a carbon/epoxy composite sheet and tested under flexure after being submitted to different environments.

Table 2.1 Test Parameters Found in Literature Review of Freeze-Thaw (F.T.) Tests

Reference	No. specimens	No. cycles	Tests Procedures	Type of FRP	Variable	Comments
DE-2	60 beams 38.1x 28.6x330mm Reinf ∅ =2.38mm Adhes. = Sikadur 32 Aggregate size = 3.175 mm	50, 100	Epoxy-concrete compatibility ASTM C884-87 Freeze-thaw (F.T.) ASTM C672-84 Flexural test (4 pts) after environmental cycling	Aramid, E-glass, Graphite (Carbon)	type of FRP: Aramid, E-glass, Graphite (Carbon)	• Graphite has higher env. durability than Aramid and E-glass. • For CFRP 21% decrease in strength compared with 17% for control beams. • Wet/dry cycles more critical than Freeze-thaw test. • Change in failure mode due to environmental exposure. • Partial debonding prior to failure.
DE-3	Field application Foulk Road bridge	(1 year)	6 beams	CFRP	- Performance during its first year. - Evaluation of bond characteristics. - Durability of joints	• All of the single layer applications of CFRP were well bonded (5 beams). • 1 beam with 2 layers of the highest strength sheet was not well bonded. Assumed due to insufficient resin saturation during the bonding process.
XX-9	2 coupons 25.4 x 9.53 x 304.8 mm	300	Cut edges coated with epoxy 2% water solution Temp = 0-40 F F.T. (ASTM C666) Flexure (ASTM D790)	Fiber glass composite	Edge coated-uncoated Salt water	• Lost 20-30% flexural strength, rigidity and toughness. • Loss due to only salt water exposure =5-10%.
XX-25	FRP rods embedded in a concrete cube of 100 mm length	300	bond test CP110 ASTM and RILEM	GFRP, CFRP and Vynylon FRP bar	Bond strength	• No great influence of the bond strength due to F. T. Cycles.

Reference	No. specimens	No. cycles	Tests Procedures	Type of FRP	Variable	Comments
XX-26	Information Not Available (N.A.)	N.A.	N.A.	CGFRP grid (74% glass, 26 % Carbon)	Range T	• Increase ultimate tensile strength by 11% and 2.5% mod. Elasticity for a fall in Temp. of -30C. • Decrease of 9.5% and 2.5% respect. for increase from room T to 50C.
XX-22	N.A.	N.A.	N.A.	N.A.	Combine env. Expos. Crack propagation	• The freezer condition had little effect on crack propagation.
CAN-1	42 columns, 15 were subjected to FT test 152 x 305 mm	200	Cycling = -18C to +20C	CFRP wrap	% of Reinf. # of CFRP layers Env. Conditions: F.T. test Low Temperature water	• CFRP wrapped concrete columns exposed to F. T. cycling showed a significant increase in strength (3 times) compared to unwrapped columns exposed to the same cond. • A second layer of CFRP provided an increase of 15% in strength. • The wrapped columns subjected to F.T. cycling failed in a more catastrophic manner than those at room Temperature.
CAN-2	18 beams	N.A.	40C one week, -23C one week for 2 months Flexural test (3pt. Bending)	N.A.	Accelerated env. Exposure	• 7% Reduction in strength for beams subjected to hot-cold cycles. • Results from hot-cold cycles are quite close to those for long term exposure. • The effect of Temperature is more important than humidity in reducing the bonding strength.
CAN-4	12 beams 102 x 152 x 1220 mm	50	F.T. cycle -18C to +20C (cold room overnight to water bath)	CFRP	- # of CFRP sheet (0,1) - Orientation of sheet (long vs. Transv.) -Freeze-thaw vs. Room Temperature	• No decrease in ultimate strength due to F.T. cycles • Strengthening with CFRP improves strength and ductility. • No difference in failure mode when compared with control beams. F.T. cycling affects cracking behavior but does not affect ultimate behavior.
CAN-5	13 cycliders 150 x 300 mm	50	F.T. cycle same as CAN-4	CFRP	-1 and 2 CFRP wraps -F.T. vs room Temp.	• CFRP wraps are effective in strengthening concrete columns after exposure to F.T. cycling

Unidirectional graphite/epoxy composite sheets were made by autoclave-vacuum molding using Newport NCT-301 composite prepeg. Thickness varied from 0.33 mm for 3 layers to 6 mm for 45 layers. Ciba-Geigy's structure epoxy adhesives AW106 and Rp1700-1 were used for the bonding procedure.

Results showed that accelerated environmental exposure by water immersion for 60 days had a slight positive effect on the load bearing capacity. Exposure to hot-cold cycles for 60 days and long term outdoor exposure up to 28 months both reduced the load bearing capacity for about 7%. The authors found that the effect of temperature was more important than humidity. Thus hot-cold cycle was presented as an effective method for accelerated test.

Reference: [CAN-3]

Baumert et al. review in this paper the existing information on the low temperature response of reinforced concrete members strengthened with FRP sheets. It reviews the material behavior of concrete, steel, and FRP at low temperatures, and discusses the observed behavior of reinforced concrete beams, with and without FRP strengthening when subjected to low temperatures.

Experiments on tensile loading of unidirectional FRP at low temperature (Dutta, 1990) have shown that the longitudinal strength of these composites drops at low temperatures. It is generally believed that in unidirectional FRP with a high fiber volume fraction tensile loading is primarily governed by the fiber properties.
To investigate the effect of freeze-thaw cycles, 6 test beams were subjected to 100 freeze/thaw cycles of 20C to -25C before being tested to failure in four point bending at room temperature (Kaiser, 1989). Half of these beams were cracked prior to adhesion of the laminate. During temperature cycling, the frozen beams were thawed by flooding the freezers with water at a temperature of approximately 20C. It was expected that water would enter into cracks and expand with subsequent freezing, resulting in peeling of the laminate. All frozen beams were brought to room temperature before being tested. A comparison of the breaking loads of the frozen beams with the breaking loads of the control beams showed no negative influence on the ultimate load capacity.
Concrete beams strengthened with FRP sheets may increase in strength when subjected to short-term exposure to low temperatures. Long-term exposure of such members must be investigated to determine the effects of creep and aging of the materials.

Reference: [CAN-4]

Soudki and Green present the results of an investigation into the effects of freeze- thaw cycling on the flexural and shear behavior of beams post-strengthened with CFRP sheets. Of twelve rectangular beams with different steel and CFRP reinforcement configuration half were finally subjected to 50 freeze-thaw cycles and half were kept in room temperature. All the beams were finally subject to a 4 point flexural test. Researchers concluded that CFRP sheets are effective in strengthening flexural members exposed to freeze-thaw cycling. CFRP sheets can be used as external shear and/or flexural reinforcement. Strengthening concrete beams with CFRP sheets improves strength and ductility. Failure modes observed were:

- bond peeling of CFRP sheet (BF).
- rupture of CFRP fibers (FD)

There was no difference in failure mode between beams subjected to freeze-thaw and those at room Temperature. Freeze-thaw slightly affected cracking behavior of beams, but does not affect ultimate behavior. Finally, the theoretical predictions compared well with test results.

Reference: [CAN-5]

Soudki and Green present the results of an investigation into the effects of freeze- thaw cycling on the efficiency of using CFRP unidirectional fiber wraps for strengthening circular columns. Thirteen plain concrete circular cylinders (150 x 300 mm) were tested under axial compression loads, seven of them were wrapped with 1 or 2 CFRP sheets. Nine cylinders were subjected to 50 F.T. cycles while the remaining four cylinders were kept at room temperature. Regarding the freeze-thaw test, the cylinders were placed in a cold room at −18C for 16 hours (overnight), they were removed in the morning and thawed in a water bath at 18C for 8 hours. Axial and circumferential strains were measured with strain gages placed on the cylinders after the completion of the 50 freeze-thaw cycles. Test results showed that the CFRP wraps enhance the axial compressive strength through confinement of the concrete in the radial direction. Considering the fact that the unwrapped specimens had a decrease on its axial strength of 46% after 50 F.T. cycles, the CFRP wrap (2 layers) was able to restore the level of strength of the unwrapped specimens at room temperature. The CFRP wraps appear to be efficient in strengthening concrete cylinders after exposure to freeze-thaw cycling in terms of strength, stiffness, and ductility. The wrapped cylinders subjected to freeze-thaw cycling failed in a more catastrophic fashion than those at room temperature.

2.1.3. EMPA - Swiss Federal Laboratories for Materials Testing and Research

Reference: [EMPA-1]

Meier reported that when a change of temperature takes place, the differences in the coefficients of thermal expansion of concrete and the carbon fiber composites resulted in thermal stresses at the joints between the two components. After 100 frost cycles ranging from +20 °C to -25°C, no negative influence on the loading capacity of three post-strengthened beams tested was found.

2.1.4. Other Research On Freeze-Thaw

Reference: [XX-9]

Gomez et al. suggested that cycles of freezing and thawing temperatures may magnify the effects of water absorption: the expansion of the freezing water could cause further delamination and interfacial failure.
Two commercially available fiber glass composite coupons were placed in a 2% salt water solution and subjected to 300 cycles of freezing and thawing, with the temperature ranging between -17.8° C and 4.4° C. Results indicates significant loss(20-30%) in flexural strength, rigidity, and toughness. For only salt water, the percentage reduction was 5-10%.

Reference: [XX-26]

Rahman et al. found that the ultimate tensile strength of Carbon-Glass Fiber Reinforced Plastic (CGFRP) grid (74% glass+26 % carbon) increased by 11 % and the modulus of elasticity by 2.5% when the temperature falls to -30° C. On the other hand, tensile strength and modulus of elasticity decreased by 9.5% and 2.5%, respectively, when the temperature rose to 50° C from the room temperature.

Reference: [XX-27]

Shulley et. al investigated the durability of five different types of reinforcing fibers (3 carbon and 2 glass) bonded on steel using the wedge-crack extension test. Different types of fibers had different durabilities against different environments. The crack growth was affected by hot water(65° C) , sea water, aqueous environment, and freeze-thaw (from -18° C to 20° C) in the order of importance of environmental effect. A sub-zero environment (-18° C) had little effect on crack growth (a slightly positive effects for some fibers).

3. EXPERIMENTAL PROGRAM

3.1. Parameters of study

A number of freeze-thaw tests were undertaken on reinforced concrete beams with glued-on CFRP plates. The freeze-thaw equipment was available at the University of Michigan. ASTM C666 procedure B was followed (see Section 3.4: Test Set-up and Instrumentation). The specimens were subjected to up to 300 freeze-thaw cycles. For every parameter, three beams were tested in bending at 0, 100, 200 and 300 cycles.

Parameters investigated were:
1) Two different adhesive systems: the Tonen system, which was selected by the technical advisory group as the primary system to be studied, and the Sika system, which represents a feasible alternative.

2) Cracking stage. A precracked state simulates the cracking conditions in the field prior to strengthening. It was expected that water would enter into the cracks during the thawing cycle and expand with subsequent freezing, resulting in a more critical situation for the performance of the strengthening system. The influence of the presence of cracks was compared with that of specimens without initial cracks.
3) Control specimens (precracked reinforced concrete beams with no CFRP laminate externally glued-on) were also subjected to 0, 100, 200 and 300 cycles.

Table 3.1. presents the summary of the parameters tested.

Table 3.1 Parameters of the Freeze-Thaw Beam Tests

Parameters	Number of Freeze-thaw cycles				Total number of specimens (48)
	0	100	200	300	
Control beam (Precracked, no CFRP glued-on)	3	3	3	3	12
Sika System (Precracked beams)	3	3	3	3	12
Tonen System (Precracked beams)	3	3	3	3	12
Tonen System (not precracked beams)	3	3	3	3	12

A total of 48 beams were tested under four-point load.

3.2. Material Properties

3.2.1. Concrete

The same type of concrete mix was utilized for the specimens and for the dummy beams that were used for calibration of the freeze-thaw chamber.

Nine mixtures were prepared. The cement used was ASTM Type III high early strength (meets MDOT Standard Specifications) and the fine aggregate had a gradation of 2NS. The coarse aggregate was provided by France Stone Company; its gradation was 26A and its Pit number was 58-9. This aggregate meets MDOT requirements regarding freeze-thaw dilation, with a dilation of 0.008% (MTM 115). The mix proportions (by weight) were:
Cement (ASTM type III) = 1.0 (418 Kg-f/m^3)
Sand (saturated, surface-dry conditions) = 1.50 (626 Kg-f/m^3)
Coarse Aggregate (saturated, surface dry conditions) = 2.54 (1062 Kg-f/m^3)
Water = 0.38 (160 Kg-f/m^3)

Two additives were used: an air entraining agent, in order to obtain a minimum air content of 6% $\pm$ 1.5 according to MDOT requirements, and a superplastisizer for better workability of the mix during pouring.

The volume of air contained in each mix was measured using a roll-a-meter. The average value obtained was 5% which falls within the admissible range. Compressive strength was obtained by testing cylinders after a curing time of at least 21 days. Average values are presented in Table 3.2.

Table 3.2 Compression Test Results

Mix number, Date	Number of cylinders	Date testing	Test data (KN)	f'c (MPa) ave.	√f'c ave.
1, October 30 1997	2	21 days	307.31-193.35	30.88	5.56
2, October 30 1997	2	21 days	250.60-183.66	26.79	5.18
3, Nov. 4 1997	3	46 days	344.28-295.57-330.04	39.88	6.32
4, Nov. 4 1997	4	46 days	166.8-215.28-290.68-288.05	29.65	5.45
5, Nov. 5 1997	3	45 days	236.63-310.25-296.90	34.70	5.89
6, Nov.5 1997	3	45 days	305.58-308.91-241.53	35.20	5.93
7, Nov. 10 1997	2	40 days	278.76-284.23	34.72	5.89
8, Nov. 10 1997	3	40 days	305.58-175.78-191.53	27.67	5.26
9, Nov. 24 1997	3	26 days	264.79-322.08-324.7	38.03	6.17

3.2.2. Steel mesh

Different possibilities were considered in order to define the appropriate amount of reinforcement (bending and shear) needed to allow precracking the beams and to guarantee shear capacity once the beams were strengthened with the CFRP laminates. A galvanized steel square welded wire mesh was chosen, with a wire spacing of 25.4 mm x 25.4 mm and a wire diameter of 1.6 mm. Tensile specimens of mesh coupons (304.8 mm long x 50.8 mm wide) were tested in order to obtain its tensile strength. Figure 3.1 presents a typical stress-displacement curve. Average tensile yield stress was found to be 400 MPa. Maximum elongation at failure, measured from machine displacement was 23 mm.

3.2.3. Strengthening Systems

3.2.3.1. Tonen System

The strengthening system was supplied by Master Builders. Its commercial name is: MBrace Composite Strengthening System. It has five components:

- MBrace Primer
- MBrace Putty filler (not used)
- MBrace Saturant Resin
- MBrace Fiber Reinforcement (MBrace CF130 Carbon fiber system)

- MBrace Topcoat (not used)

MBrace Putty filler was not used since it is intended to be used to patch cracks in old concrete and the concrete used did not required this surface preparation. MBrace Topcoat is an optional finishing layer for painting appearance and UV protection. Since the testing of the beams was to be performed indoors, this finishing was not used in this experimental program.

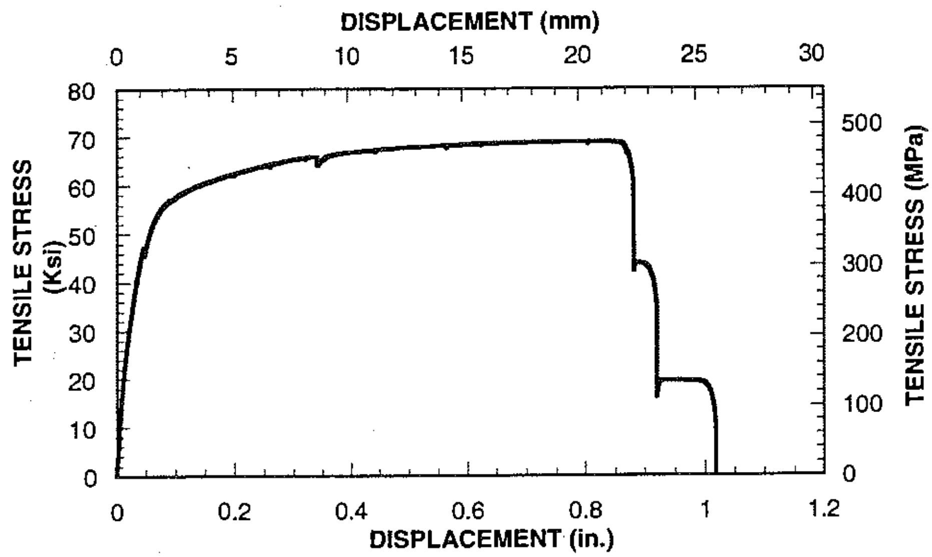


Figure 3.1 Stress-Displacement Curve of the Steel Mesh

Typical Properties of MBrace CF 130 (provided by the supplier):

Fiber Reinforcement: Carbon Fiber, High Tensile
Fiber Density: 1.82 g/cm^3
Fiber Modulus: 2.35 x10^6 kg-force/cm^2
Fiber Areal Weight Density: 300g/m^2
Sheet Width: 50 cm
Tensile Strength: 590 kg-f/cm-sheet width
35,500 kg-f/cm^2
Tensile Modulus: 38,800 kg-f/cm-sheet width
2.35 x10^5 kg-f/cm^2
Design Thickness: 0.165 mm/ply
Tensile Elongation at ultimate: 1.5%

Typical Properties of MBrace Saturant (i.e., epoxy resin as called by the supplier):

Volatile Organic Compounds: 20 g/liter
Flash Point: 72°C
Mixed Viscosity @ 20°C: 1,600 cps
Color: Blue
Weight/Gallon: 1.04±0.024 kg/L
Shelf Life @ 20°C : 18 months

Flexural Strength: 43 MPa
Tensile Strength: 78 MPa
Compressive Strength: 88 MPa
Work Time @ 20°C : 30 minutes

Typical Properties of MBrace Primer (i.e., primer resin as called by the supplier):

Generic type: Amine-cured liquid epoxy
Solids content: 100%
Color: clear Amber
Weight/Gallon: Part A 1139g/L
Part B 996 g/L
Tensile Strength: 13 to 15.8 MPa
Tensile Modulus (Tangent): 689 to 826.8 MPa
Tensile elongation: 20-30%
Tensile bond strength (steel): 17 MPa
Work Time @ 20°C : 45 hours

3.2.3.2. Sika System

The Sika Company provided the Carbodur strengthening system. Components of this system are:

- Sika Carbodur CFRP (Carbon fiber laminate strips).
- Sikadur 30 (epoxy adhesive).

Typical Properties of Sika CFRP Strips (provided by the supplier):

Tensile Strength: 2,400 MPa
Modulus of Elasticity: 150 x10^3 MPa
Density: 1.6 g/cm^3
Thickness: 1.2 mm
Sheet width: 50 or 80 mm
Elongation at ultimate: 1.4%

Typical Properties of Sikadur 30(provided by the supplier):

Application Temperature: 18-30°C
Pot Life @23°C : 70 min
Compressive Strength (14 day) >58.6 MPa
Shear Strength: 24.8 MPa
Tensile Strength @7 day: 24.8 MPa
Elongation at break: 1%

3.3. Design and Fabrication of Specimens

Typical freeze-thaw test of concrete beams according to ASTM C-666 requires beams of dimensions 76 mm x 76 mm x 406 mm (ASTM C666). For this particular freeze-thaw test and considering that the beams were to be tested under flexural loads, the dimensions of the reinforced concrete beams for the freeze-thaw tests was 76 mm x 76 mm x 1016 mm, see Figure 3.2.

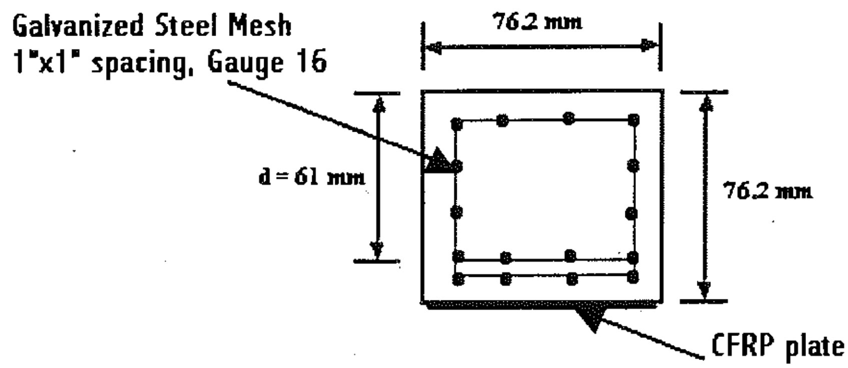


Figure 3.2 Reinforcement Detail of the Cross Section

Prior investigators ([X-27], [EMPA-1], [CAN-3], [CAN-4]) have indicated that the freeze-thaw cycles had little effect on the CFRP laminate itself. However, the bond strength at the interface between the concrete and the CFRP seems to be the controlling parameter.

The reinforced concrete beams were designed with enough longitudinal reinforcement to support the precracking load without failure. The width and length of the CFRP sheet for both systems was calculated assuming that once the interface between the concrete and the CFRP reaches the value of the shear bond strength of the concrete ($0.17\sqrt{f'c}$), peeling off of the CFRP laminate will occur, see Appendix A. A width of 76.2 mm for the CFRP was used for the Tonen system and 80 mm was used for the Sika system.

For the Tonen system, a minimum value of development length of 371 mm was found to be necessary in order to prevent interfacial failure between the CFRP sheet and the concrete surface, leading to a peeling-off of the CFRP. However because the objective of this project was to evaluate the influence of the freeze-thaw cycles on bond, a shorter length was selected namely 254 mm as the development length for the CFRP sheet. The total length of the CFRP was 660 mm (see Figure 3.3).

For the Sika system, the minimum value of development length calculated was 872 mm. Following the same line of though presented above, a smaller value (356 mm) was adopted. The total length of the CFRP was 864 mm.

The moment capacity for each system was different since the Sika laminate provides almost 5 times the tensile load of the Tonen laminate per unit width. From Appendix A it can be seen that the moment capacity of the reinforced concrete beam strengthened with the Sika plate system is 1.7 times the moment capacity for the reinforced concrete beam strengthened with the Tonen sheet system. This difference will be considered in the analysis of the test results.

For the fabrication of the specimens, plexiglass molds were utilized in order to obtain a smooth surface finish. The steel mesh was placed inside the molds leaving a 13 mm cover around the perimeter of the cross section (see Figure 3.2). Thermocouples were installed in three dummy beams and three specimens. Once the specimens were installed in the freeze-thaw chamber, the thermocouples provided information to the computerized controller to adjust the conditions inside the freeze-thaw chamber in order to meet the target profile.

The concrete was mixed in a laboratory concrete mixer. All specimens were removed from their molds 24 hours after pouring, and were placed in a water tank for 21 days. Specimens that were placed in the freeze-thaw chamber had at least 3 weeks of age.

3.4. Test Set-Up and Instrumentation

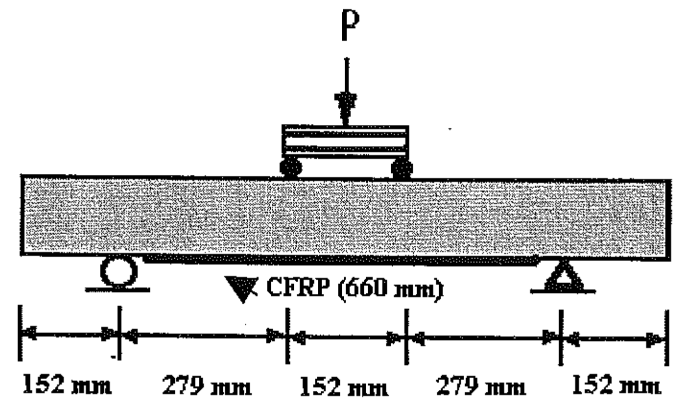
3.4.1. Freeze-Thaw Cycles

As mentioned in the introduction, the freeze-thaw equipment available at the University of Michigan allowed exposure as per ASTM C666, Procedure B requirements. The freeze-thaw machine was programmed to run freeze-thaw cycles according to the Michigan Test Method for Testing Concrete for Durability by Rapid Freezing in Air and Thawing in Water (MTM 115-97). This method conforms to the general requirements of ASTM C666, Procedure B.

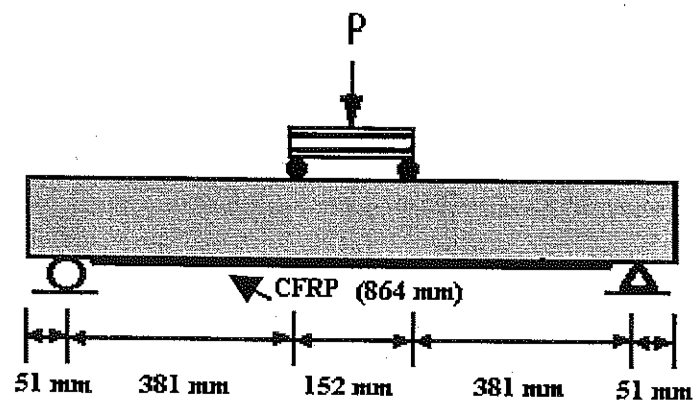
The nominal freezing and thawing cycle for this method consisted of alternately lowering the temperature of the specimens from 4.4°C to -17.8°C and raising it from -17.8°C to 4.4°C within the temperature limitations of ASTM C666. The nominal cycle length was 3 hours. Figure 3.4 shows a schematic representation of the freeze-thaw test.

The freeze-thaw machine was programmed with a profile defined to follow, as close as possible, the nominal freezing and thawing cycle adopted by MDOT. The profile was tried using 24 dummy beam specimens. Once the profile was tested with the dummy beams and showed compliance with MDOT requirements, the "real" specimens were tested. Figure 3.5 presents the readings from the freeze-thaw machine after its calibration.

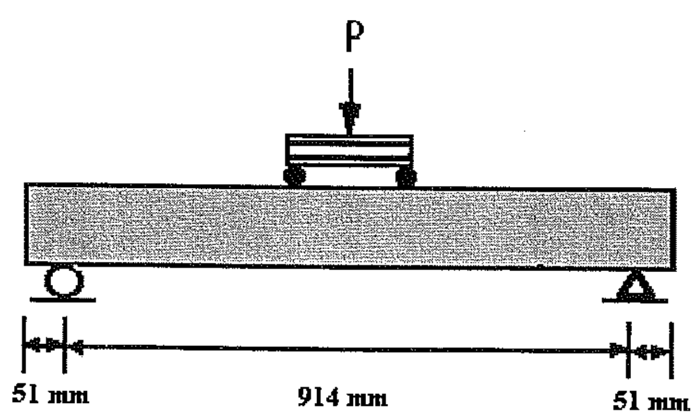
After 100, 200 and 300 number of freeze-thaw cycles, 12 specimens were removed each time from the chamber and placed into a water tank at room temperature until the time of flexural testing.



Tonen System: Load Set-Up



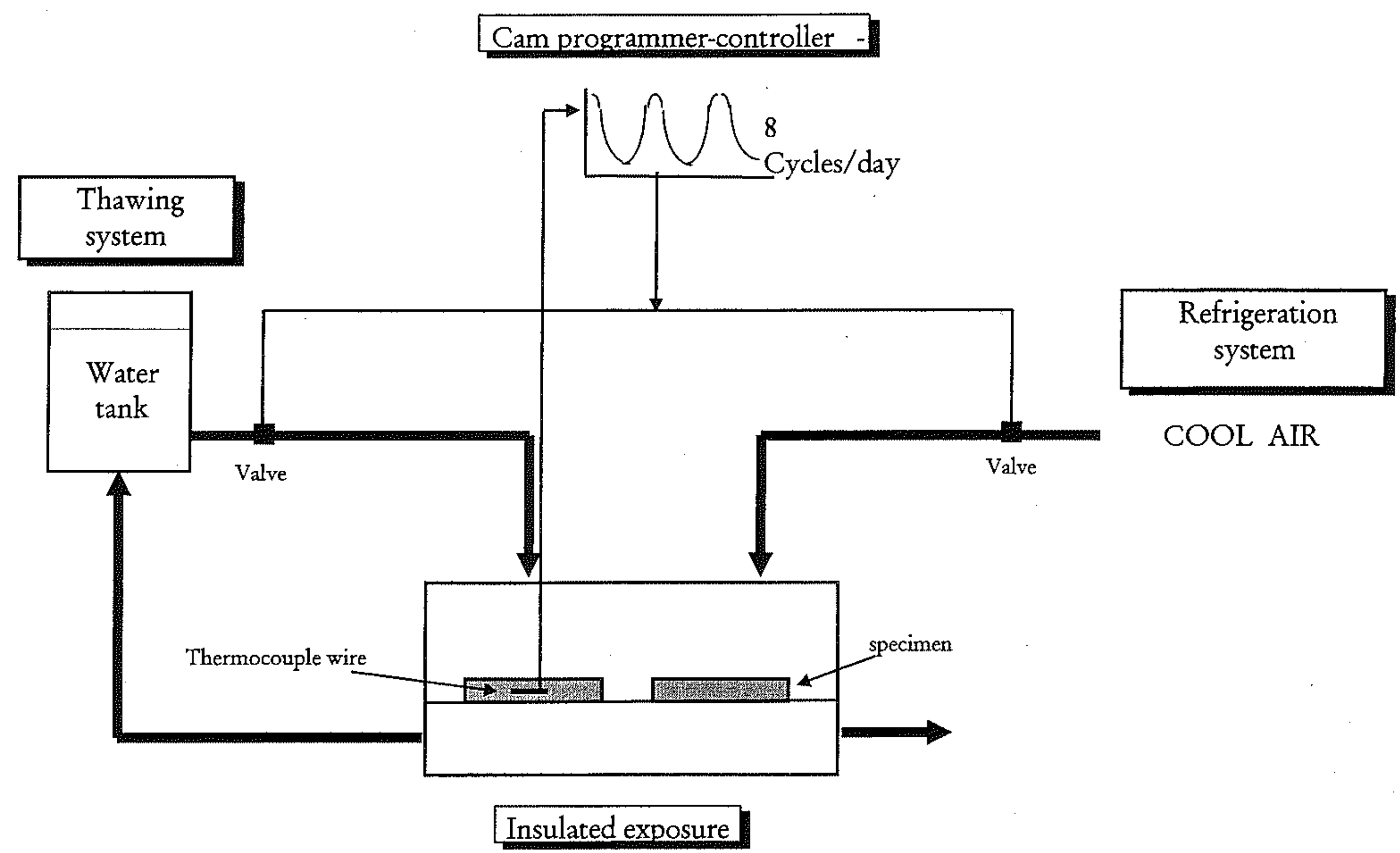
Sika System: Load Set-Up



Control Beams: Load Set-Up

Figure 3.3 Flexural Test Load Set-up

Figure 3.4 Schematic Representation of the Freeze-Thaw Test (ASTM C666-procedure B)



The use of the water tank was suggested by the TAG to guarantee 100% wet conditions at the time of testing. The age of the specimens at the time of the flexural test was at least 4 weeks.

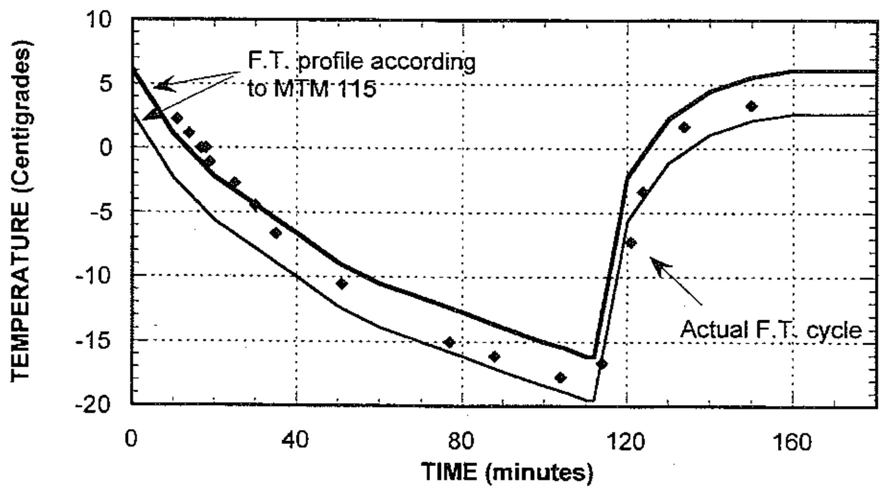


Figure 3.5 Freeze-Thaw Profile Applied

3.4.2. Bending Test

All the specimens were tested at room temperature under four-point load in an INSTRON testing Machine, Model 4206, with a load cell of 133,000 N capacity. The specimens were tested under displacement control. The rate of loading was 0.25 mm/min. Applied loads and displacements were recorded from the load cell and the crosshead of the machine. The bending tests were also video taped. Pictures were taken during and at the end of each test. Some photos of failure modes are shown in appendix D.

4. TEST RESULTS

A total of 48 beams were tested under 4 point bending. The applied loads versus displacements were recorded and the type of failure observed.

Table 4.1 presents a summary of these results: Peak load, deflection at failure, and type of failure are reported for each beam tested. Table 4.2 presents the values of maximum shear force and maximum moment at failure.

Figures 4.1, 4.2, 4.3 and 4.4 present typical load-displacement curves for the Sika and Tonen systems at 0, 100, 200 and 300 freeze-thaw cycles, respectively. Figure 4.5 to 4.7 present load-displacement curves for the Tonen precracked, Tonen not precracked and Sika precracked beams.

Table 4.1 Summary of the Test Results

Number of Cycles	Parameter	Peak Load (KN)	Deflection at Failure (mm)	Delamination Length (mm)	Type of Failure
0 cycles	No CFRP (precracked beam)	2.65	7.62	0.00	Yield and fracture of reinforcement
		2.52	7.11	0.00	Yield and fracture of reinforcement
		2.64	8.03	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	19.54	10.85	101.60	Shear-delamination (crack at 30°angle)
		20.49	12.37	279.40	Shear-delamination.
		19.90	11.15	69.85	Shear-delamination.
	Tonen sheet (non-precracked)	16.79	9.50	196.85	Flexure-delamination
		17.81	9.96	165.10	Flexure-delamination
		17.04	9.55	139.70	Flexure-delamination
	Sika sheet (precracked beam)	22.02	8.76	0.00	Shear (crack length = 178 mm)
		21.29	8.10	0.00	Shear
		23.99	9.58	254.00	Shear-delamination
100 cycles	No CFRP (precracked beam)	2.85	7.62	0.00	Yield and fracture of reinforcement
		2.81	7.11	0.00	Yield and fracture of reinforcement
		2.73	9.65	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	16.09	9.65	127.00	Shear-delamination (45°angle)
		15.81	9.91	152.40	Shear-delamination
		15.93	9.65	165.10	Shear-delamination-flexure (50°angle)
	Tonen sheet (non-precracked)	16.46	10.16	152.40	Shear-delamination
		16.46	9.40	177.80	Shear-delamination
		10.98	8.64	139.70	Shear-delamination
	Sika sheet (precracked beam)	19.55	7.11	0.00	Shear (crack length =229 mm)
		20.39	7.87	0.00	Shear (crack length = 229 mm)
		20.18	7.62	0.00	Shear
200 cycles	No CFRP (precracked beam)	2.94	6.86	0.00	Yield and fracture of reinforcement
		2.96	6.86	0.00	Yield and fracture of reinforcement
		2.53	7.62	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	15.40	6.10	139.70	Shear-delamination (40°angle)
		13.34	6.86	0.00	Shear-delamination
		13.78	7.62	215.90	Shear-delamination (60°angle)
	Tonen sheet (non-precracked)	14.06	7.37	139.70	Flexure-delamination
		13.34	7.37	177.80	Shear-delamination (45°angle)
		14.44	7.62	203.20	Flexure-delamination (75°angle)
	Sika sheet (precracked beam)	19.57	7.62	203.20	Delamination-flexure (25°angle)
		19.64	7.37	241.30	Delamination-shear (45°angle)
		19.49	7.62	0.00	Shear (crack length = 203 mm)
300 cycles	No CFRP (precracked beam)	2.94	6.35	0.00	Yield and fracture of reinforcement
		2.49	6.35	0.00	Yield and fracture of reinforcement
		2.75	6.10	0.00	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	12.66	6.86	177.80	Flexure-delamination
		12.45	6.35	165.10	Flexure-delamination
		12.13	6.35	190.50	Shear-delamination (40°angle)
	Tonen sheet (non-precracked)	13.22	7.87	152.40	Shear-delamination
		14.52	7.62	177.80	Shear-delamination (35°angle)
		13.74	8.13	0.00	Shear-delamination
	Sika sheet (precracked beam)	21.88	8.64	177.80	Shear-delamination (35° angle)
		21.33	8.38	241.30	Shear-delamination flexure (25°angle)
		21.14	7.62	254.00	Flexure-delamination (30° angle)

Table 4.2 Maximum Shear Force and Moment

Number of Cycles	Parameter	Shear Force (KN)	Max. Moment (KN-m)	Type of Failure
0 cycles	No CFRP (precracked beam)	1.33	0.51	Yield and fracture of reinforcement
		1.26	0.48	Yield and fracture of reinforcement
		1.32	0.50	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	9.77	2.73	Shear-delamination (crack at 30°angle)
		10.25	2.86	Shear-delamination.
		9.95	2.78	Shear-delamination.
	Tonen sheet (non-precracked)	8.40	2.35	Flexure-delamination
		8.90	2.49	Flexure-delamination
		8.52	2.38	Flexure-delamination
	Sika sheet (precracked beam)	11.01	4.20	Shear (crack length = 178 mm)
		10.65	4.06	Shear
		12.00	4.57	Shear-delamination
100 cycles	No CFRP (precracked beam)	1.42	0.54	Yield and fracture of reinforcement
		1.41	0.54	Yield and fracture of reinforcement
		1.37	0.52	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	8.05	2.25	Shear-delamination (45°angle)
		7.91	2.21	Shear-delamination
		7.97	2.23	Shear-delamination-flexure (50°angle)
	Tonen sheet (non-precracked)	8.23	2.30	Shear-delamination
		8.23	2.30	Shear-delamination
		5.49	1.55	Shear-delamination
	Sika sheet (precracked beam)	9.78	3.72	Shear (crack length = 229 mm)
		10.19	3.88	Shear (crack length = 229 mm)
		10.09	3.85	Shear
200 cycles	No CFRP (precracked beam)	1.47	0.56	Yield and fracture of reinforcement
		1.48	0.56	Yield and fracture of reinforcement
		1.27	0.48	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	7.70	2.15	Shear-delamination (40°angle)
		6.67	1.86	Shear-delamination
		6.89	1.93	Shear-delamination (60°angle)
	Tonen sheet (non-precracked)	7.03	1.96	Flexure-delamination
		6.67	1.86	Shear-delamination (45°angle)
		7.22	2.02	Flexure-delamination (75°angle)
	Sika sheet (precracked beam)	9.79	3.73	Delamination-flexure (25°angle)
		9.82	3.74	Delamination-shear (45°angle)
		9.75	3.71	Shear (crack length = 203 mm)
300 cycles	No CFRP (precracked beam)	1.47	0.56	Yield and fracture of reinforcement
		1.25	0.47	Yield and fracture of reinforcement
		1.37	0.52	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	6.33	1.77	Flexure-delamination
		6.22	1.74	Flexure-delamination
		6.07	1.69	Shear-delamination (40°angle)
	Tonen sheet (non-precracked)	6.61	1.85	Shear-delamination
		7.26	2.03	Shear-delamination (35°angle)
		6.87	1.92	Shear-delamination
	Sika sheet (precracked beam)	10.94	4.17	Shear-delamination (35° angle)
		10.68	4.07	Shear-delamination flexure (25°angle)
		10.57	4.02	Flexure-delamination (30° angle)

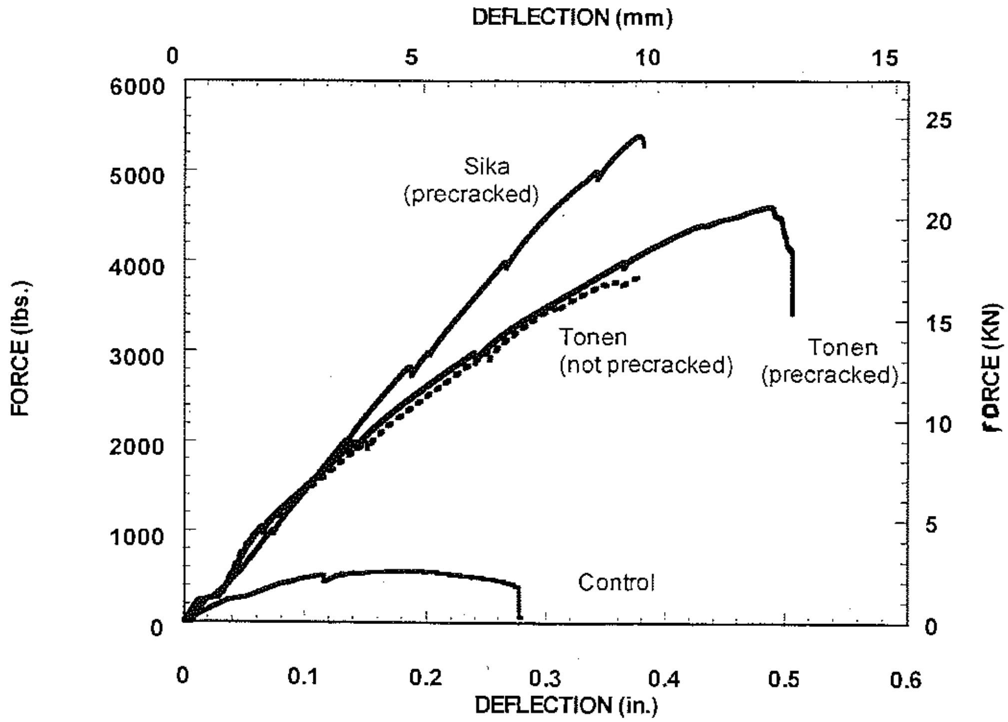


Figure 4.1 Load-Deflection Curves for 0 F.T. Cycles

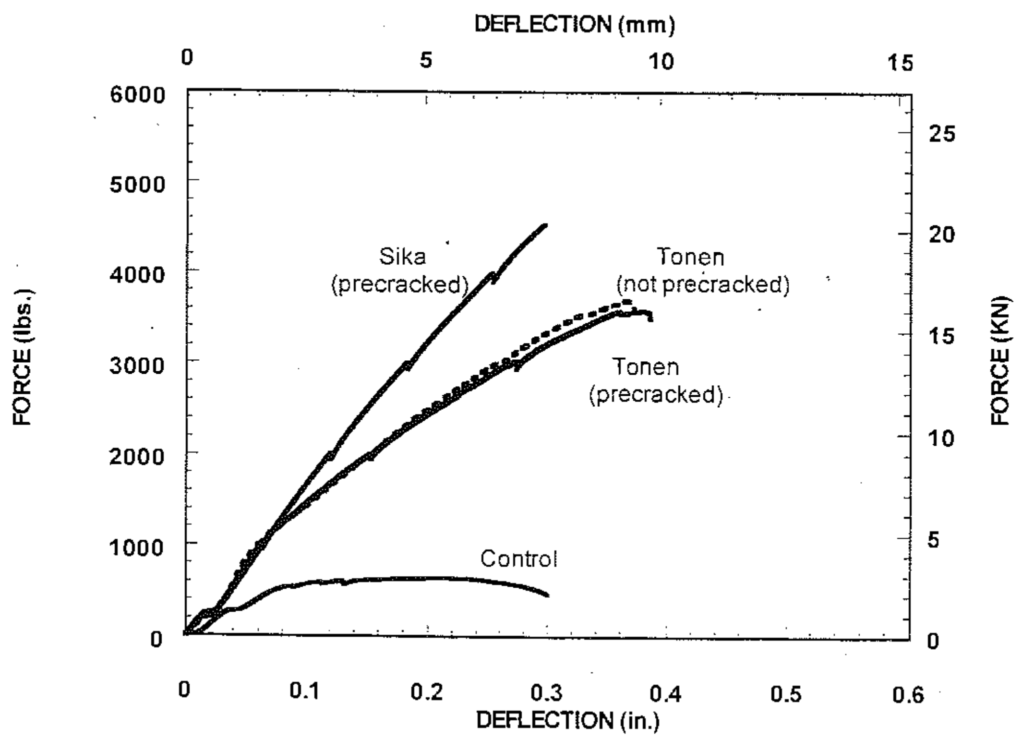


Figure 4.2 Load-Deflection Curves for 100 F.T. Cycles

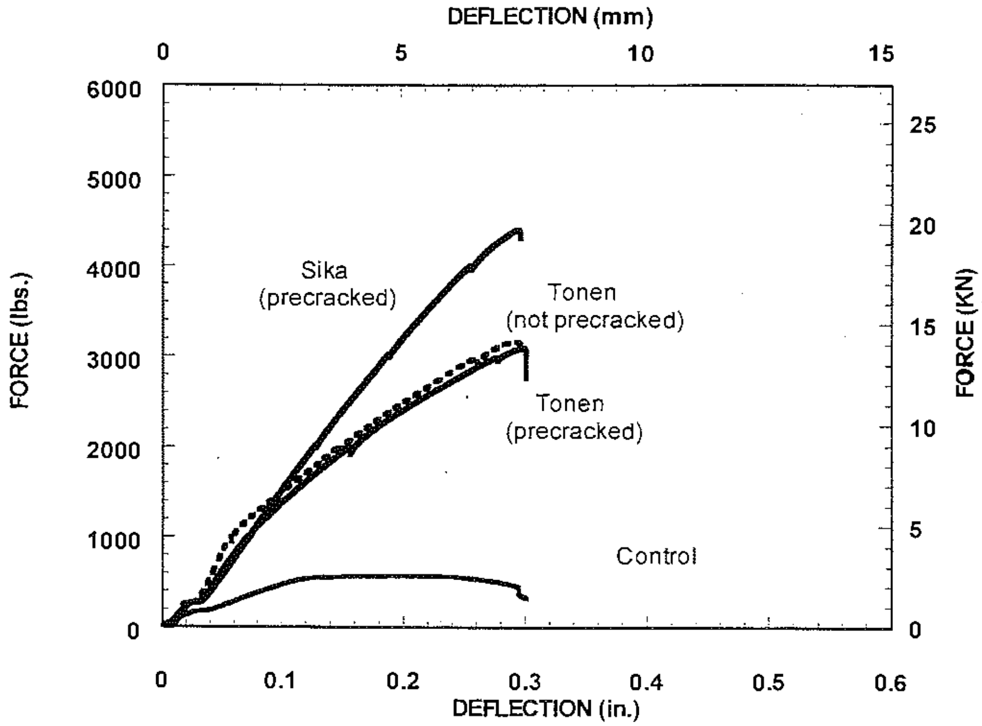


Figure 4.3 Load-Deflection Curves for 200 F.T. Cycles

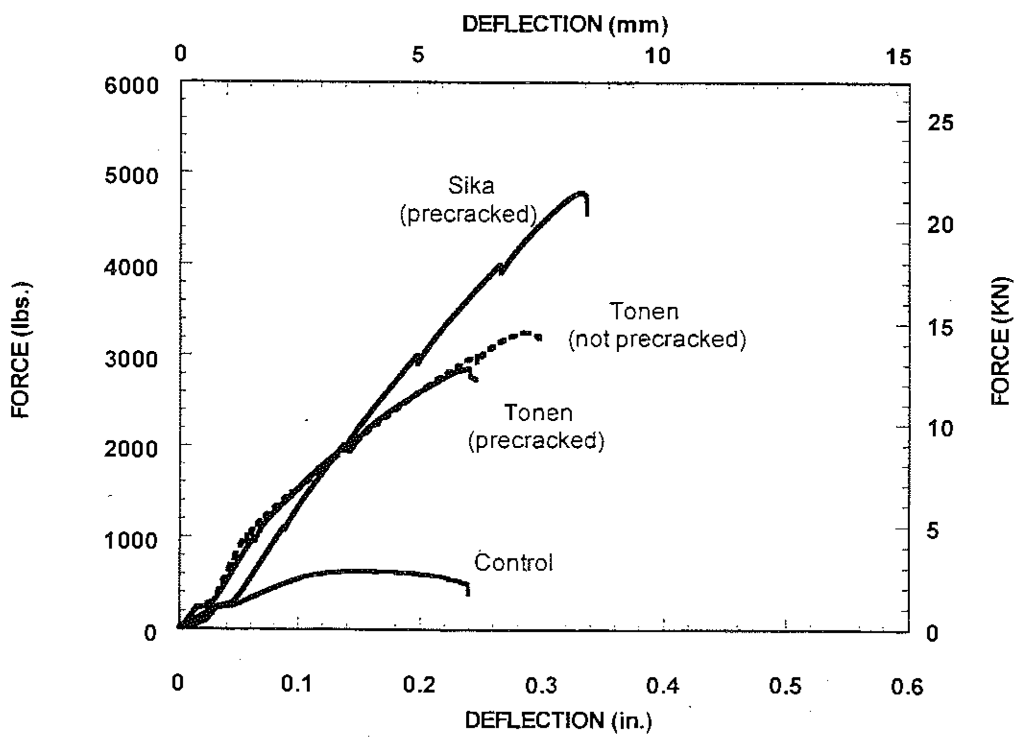


Figure 4.4 Load-Deflection Curves for 300 F.T. Cycles

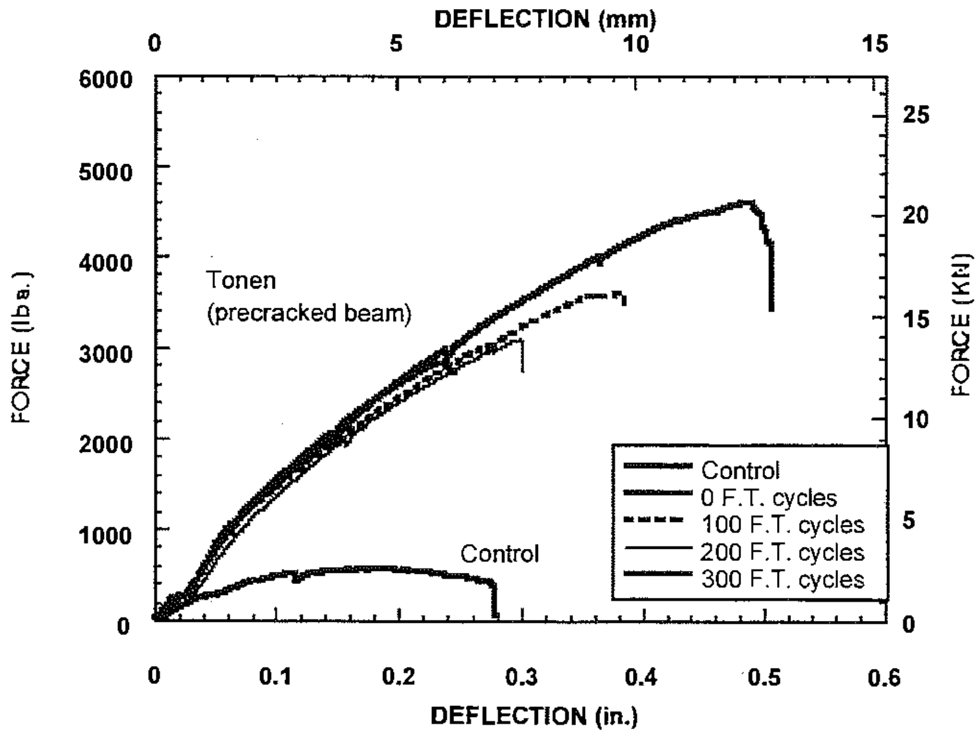


Figure 4.5 Load-Deflection Curves for Tonen Precracked Beams

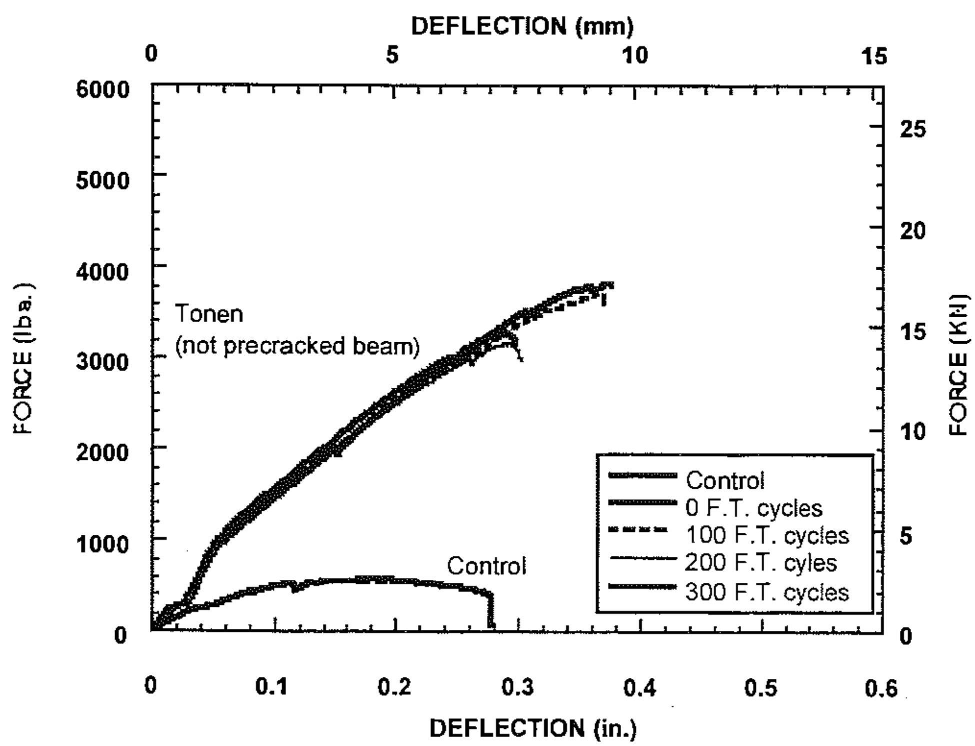


Figure 4.6 Load-Deflection Curves for Tonen Not Precracked Beams

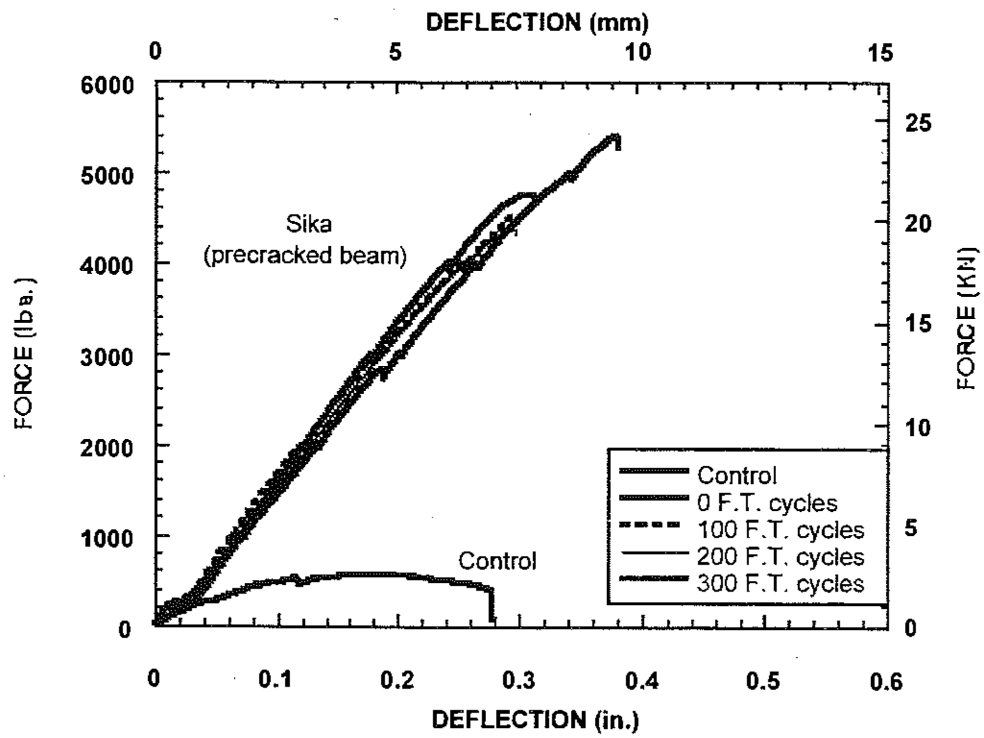


Figure 4.7 Load-Deflection Curves for Sika Precracked Beams

5. ANALYSIS AND INTERPRETATION

As was indicated in section 3.3 the specimens were designed in a way to make failure by debonding of the CFRP likely. Therefore, beams reinforced by the Tonen and the Sika system had a different reinforcement ratios and were expected to have different levels of moment capacity but failure was expected to occur by the CFRP peeling-off. Both delamination length and the corresponding type of failure observed in each test are analyzed.

It is expected that this analysis will indicate how the bond strength and development length of the CFRP laminates are affected by the number of freeze-thaw cycles.

5.1. Failure Modes

Four different types of failure mode were found during the testing of the 48 beams:

1. Yielding and immediate fracture of the steel mesh reinforcement at the tension level of the beam. This type of failure is characteristic of under-reinforced beams (control beams with no CFRP).
2. The CFRP delaminates at the interface with the concrete surface. This interfacial failure started at a flexural crack (flexure-delamination).
3. CFRP delaminates at the interface with the concrete surface. This interfacial failure started at a shear crack (shear-delamination).
4. Vertical shear failure of the reinforced beam at the end of the CFRP laminate (shear).

Figure 5.1 shows the different types of failure mode. Table 5.1 also presents the type of failure observed for each specimen tested.

For the Tonen precracked beams, a very uniform pattern was observed. All the specimens at 0, 100 and 200 F.T. cycles failed by shear-delamination. For 300 F.T. cycles, two beams failed by flexure-delamination and one failed by shear delamination.

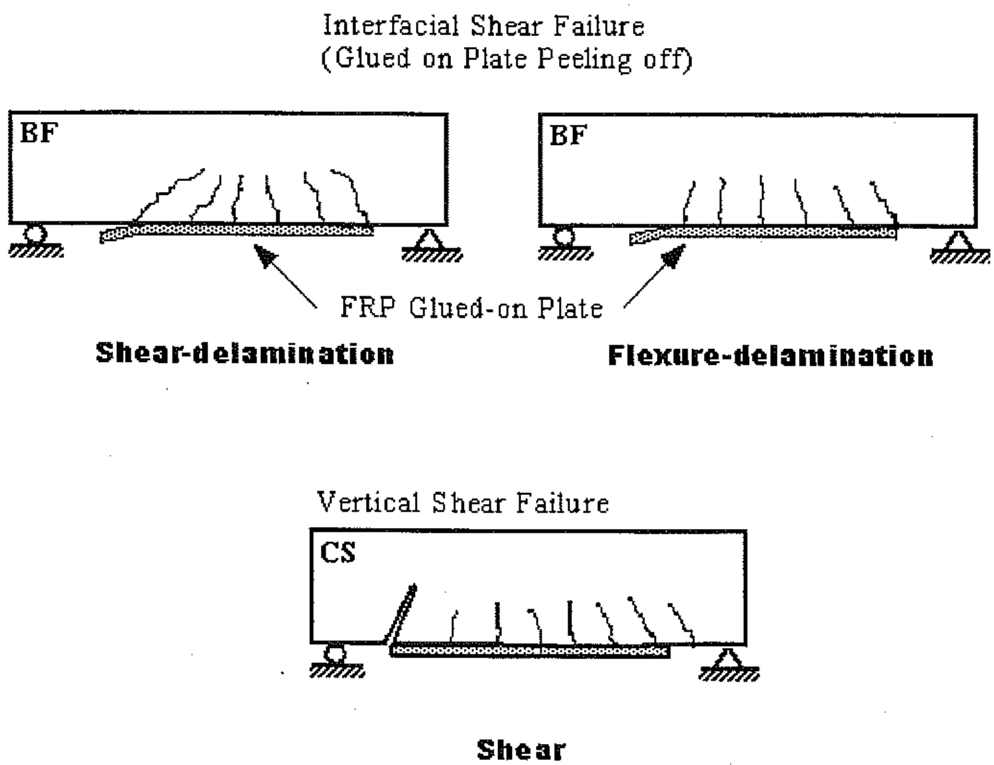
For the Tonen non-precracked beams, the type of failure was also consistent, all the specimens failed by delamination. For all the specimens at 0 F.T. cycles, flexure-delamination occurred. For the specimens at 100 and 300 F.T. cycles, shear-delamination occurred. At 200 F.T. cycles, two beams failed by flexure-delamination and one failed by shear-delamination.

For the Sika precracked beams, the pattern was not as uniform. For the specimens at 0 F.T. cycles, two beams failed by shear and one beam failed by shear-delamination. At 100 F.T. cycles, all the beams failed by shear. For the specimens at 200 F.T. cycles, each beam failed in a different failure mode (shear, shear-delamination, flexure-delamination). Finally, for the specimens at 300 F.T. cycles, two beams failed by shear-delamination and one by flexure-delamination.

It was important to find a possible correlation between each type of failure and the strength of the beams strengthened with the Sika or the Tonen system. It was also important to discover the influence of the number of freeze-thaw cycles on the bond strength.

Table 5.1 Values of Normalized Maximum Shear Stress

Number of cycles	Parameter	Shear Force (KN)	√f'c (√MPA)	Normalized Shear Stress V_n	Type of Failure
0 cycles	No CFRP (precracked beam)	1.33	5.89	0.04	Yield and fracture of reinforcement
		1.26	5.18	0.04	Yield and fracture of reinforcement
		1.32	5.89	0.04	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	9.77	5.18	0.32	Shear-delamination (crack at 30°angle)
		10.25	5.93	0.30	Shear-delamination.
		9.95	5.18	0.33	Shear-delamination.
	Tonen sheet (non-precracked)	8.40	6.17	0.23	Flexure-delamination
		8.90	6.17	0.25	Flexure-delamination
		8.52	6.17	0.24	Flexure-delamination
	Sika sheet (precracked beam)	11.01	5.89	0.32	Shear (crack length = 178 mm)
		10.65	5.18	0.35	Shear
		12.00	5.18	0.40	Shear-delamination
100 cycles	No CFRP (precracked beam)	1.42	5.93	0.04	Yield and fracture of reinforcement
		1.41	5.93	0.04	Yield and fracture of reinforcement
		1.37	5.89	0.04	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	8.05	5.89	0.24	Shear-delamination (45°angle)
		7.91	5.89	0.23	Shear-delamination
		7.97	5.89	0.23	Shear-delamination-flexure (50°angle)
	Tonen sheet (non-precracked)	8.23	5.89	0.24	Shear-delamination
		8.23	5.89	0.24	Shear-delamination
		5.49	5.89	0.16	Shear-delamination
	Sika sheet (precracked beam)	9.78	5.89	0.29	Shear (crack length = 229 mm)
		10.19	5.93	0.30	Shear (crack length = 229 mm)
		10.09	5.93	0.29	Shear
200 cycles	No CFRP (precracked beam)	1.47	5.44	0.05	Yield and fracture of reinforcement
		1.48	5.44	0.05	Yield and fracture of reinforcement
		1.27	5.44	0.04	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	7.70	5.56	0.24	Shear-delamination (40°angle)
		6.67	6.17	0.19	Shear-delamination
		6.89	5.56	0.21	Shear-delamination (60°angle)
	Tonen sheet (non-precracked)	7.03	5.44	0.22	Flexure-delamination
		6.67	5.44	0.21	Shear-delamination (45°angle)
		7.22	5.44	0.23	Flexure-delamination (75°angle)
	Sika sheet (precracked beam)	9.79	5.44	0.31	Delamination-flexure (25°angle)
		9.82	5.18	0.33	Delamination-shear (45°angle)
		9.75	5.44	0.31	Shear (crack length = 203 mm)
300 cycles	No CFRP (precracked beam)	1.47	6.31	0.04	Yield and fracture of reinforcement
		1.25	6.31	0.03	Yield and fracture of reinforcement
		1.37	5.44	0.04	Yield and fracture of reinforcement
	Tonen sheet (precracked beam)	6.33	5.56	0.20	Flexure-delamination
		6.22	5.56	0.19	Flexure-delamination
		6.07	5.56	0.19	Shear-delamination (40°angle)
	Tonen sheet (non-precracked)	6.61	5.89	0.19	Shear-delamination
		7.26	6.31	0.20	Shear-delamination (35°angle)
		6.87	5.89	0.20	Shear-delamination
	Sika sheet (precracked beam)	10.94	5.18	0.36	Shear-delamination (35° angle)
		10.68	5.18	0.36	Shear-delamination flexure (25°angle)
		10.57	5.18	0.35	Flexure-delamination (30° angle)



Notes:
Flexure-delamination: CFRP peeling-off started at a flexural crack
Shear-delamination: CFRP peeling-off caused by shear cracks
Shear: Shear failure of the reinforced beam at the end of the CFRP laminate

Figure 5.1 Failure Modes for Specimens with CFRP Plates

5.2. Shear Strength

The design calculations for the specimens showed proximity of the shear failure load to the CFRP peeling-off (delamination) load, particularly with the Sika system. It was expected that if the specimen did not fail by delamination, shear failure would immediately follow.

As stated before, for the control specimens the failure was by fracture of the steel mesh following yielding of the reinforcement. This type of failure was in accordance with the under reinforced nature of the beam. The values obtained for shear force for the control specimens were very close (average = 1.37 KN), showing them to be indifferent to the influence of the number of freeze-thaw cycles. However, a decrease of 17% in the deflection at failure was observed after 300 freeze-thaw cycles.

For the strengthened specimens, the first step in the analysis process was to compare the values of maximum shear force and displacement versus number of freeze-thaw cycles. For the Tonen system (precracked and non-precracked beams) it was clear that there was a decrease in the value of the shear force as well as the maximum deflection with an increase in the number of freeze-thaw cycles. However, this behavior did not seem to be so obvious for the Sika specimens. Contribution of the different types of failure modes and the strength of the concrete of each mix should be evaluated more accurately such as is presented next.

By the traditional theory for *homogeneous, elastic, uncracked* beams, shear stresses, v, can be calculated using the equation:

$$v = \frac{V \times Q}{I \times b} \qquad (1)$$

where V is the vertical shear force on the cross section; Q is the moment of inertia of the cross section, Q is the first moment about the neutral axis of the part of the cross-sectional area above the point where the shear stresses are being calculated and b is the width of the member. However, because the reinforced concrete beam at the failure load is beyond the elastic range and it is a cracked section, equation 1 is not accurate to calculate shear stresses.

ACI design equation (11-3) computes the shear strength provided by the concrete as:

$$V_c = 0.17\sqrt{f'c}\, bd \quad \text{(SI units)} \qquad (2)$$

where V is the vertical shear force, b the thickness of the web and d is the distance of the area of tensile steel to the maximum compression strain. ACI considers the shear strength to be based on an average shear stress on the full effective cross section b*d (see commentary R11.1). The average shear stress of the concrete is therefore equal to 0.17 $\sqrt{f'c}$. Based on this idea, average shear stress can be taken as:

$$v = \frac{V}{b \times d} \qquad (3)$$

However, since the CFRP has a significant contribution to the force equilibrium of the section, the real position of "d" is located near the centroid of the laminate. Therefore, the use of "h" instead of "d" in the calculation of the v will give a more accurate evaluation of the shear stress:

$$v = \frac{V}{b \times h} \qquad (4)$$

In order to eliminate the contribution of the strength of the concrete mix, normalized shear stresses (v_n) were calculated from the original data. The shear force obtained from the experimental data was divided by the square root of the compressive strength ($\sqrt{f'c}$) times the width (b= 76.2 mm) times the height of the beam (h=76.2 mm):

$$v_n = \frac{V}{b \times h \times \sqrt{f'c}} \qquad (5)$$

In the calculation of v_n , v was normalized with respect to f'c in order to be able to compare its value with the assumed design shear strength of concrete (0.17$\sqrt{f'c}$, MPa). Table 5.1 presents the values of normalized shear stresses for each specimen. Figures 5.2, 5.3, and 5.4 show the normalized shear stress at failure versus number of freeze-thaw cycles for each strengthening system. Figure 5.5 compares the average values of normalized shear stresses at failure.

For the Tonen System, the normalized maximum shear stress (v_n) decreased with an increase number of freeze-thaw cycles. For the beams with precracking , the decrease in stress was more significant (26% for 100 F.T. cycles, 32% for 200 F.T. cycles, and 39% for 300 F.T. cycles on the average) than for the non cracking beams (11% for 100 F.T. cycles, 9% for 200 F.T. cycles, and 22% for 300 F.T. cycles).

For the Sika system a more irregular pattern was presented. Comparing the average value of normalized maximum shear stress at 0 freeze-thaw cycles, a decrease of 18% was obtained for 100 F.T. cycles, 12% for 200 F.T. cycles, and 0% for 300 F.T. cycles. It is expected that a more consistent pattern will be obtained if discrimination is made with respect to the type of failure.

For all the tests observed, values were higher than 0.17 for the normalized shear stress.

As is shown in appendix B, this normalized shear stress is only an indirect measure of the value of horizontal interfacial shear stress at the level of the CFRP laminate. However, in order to find the exact value of the interfacial shear stress (τ) that leads to delamination failure, another set of experiment should be carried out. The effect of freeze-thaw cycles and the shear capacity of the specimens are variables that influence interfacial shear strength and should be weighted. For this report, the normalized shear stress (v_n) will be used to measure indirectly the influence of the freeze-thaw cycles on the bond strength of the specimens tested.

Normalized shear stresses decreased with an increasing number of freeze-thaw cycles for the Tonen precracked beams. Maximum and minimum values of normalized shear stress for the specimens were 0.32 and 0.19. For the Tonen system without precracking, the maximum and minimum values of normalized shear stress were closer: 0.25 and 0.19.

For the Sika system with precracked beams, maximum and minimum values of normalized shear stress were 0.40 and 0.29.

For the control specimens, the average value of normalized shear stress was 25% of the design value and was indifferent to the number of freeze-thaw cycles. This low value was related to the type of failure (yielding of reinforcement) showing that the control beams did not develop their maximum capacity in shear.

It can be concluded that the values of normalized shear stress were higher than 0.19 for the Tonen system and 0.29 for the Sika system. In no case was the value of 0.40 surpassed.

Yoshizawa et al. [FTS-5] have found that the bond strength of the CFRP sheets increases with an increase in the number of CFRP layers. From appendix A, preliminary design of the beam capacity with Tonen and Sika system indicated that the moment capacity of the beam with Sika system was 1.7 higher than the moment capacity of the beam with the Tonen system. From the experimental data analyzed above, it was found that for each number of F.T. cycles, the values of normalized shear stress for the Sika system were always higher than the ones for the Tonen system, corroborating Yoshizawa et al.' statement [FTS-5].

Considering the different failure modes, it was observed that for the Tonen system with precracking the values of normalized shear stress were higher for the flexure-delamination failure than for the shear-delamination for the same number of freeze-thaw cycles, see Figure 5.2. This result can be explained if it is assumed that in the flexure-delamination failure, the CFRP laminate can still carry tensile load between the flexural cracks whereas for the shear-delamination failure, the effect of a diagonal crack accelerate the interfacial crack propagation. Researchers at Ibaraki University [XX-46] described a similar phenomenon. However, the overall pattern of decrease of strength was followed no matter what type of failure was considered. It can be concluded that for the Tonen system with precracking, an increasing number of freeze-thaw cycles led to a decrease in the shear capacity of 39% for 300 F.T. cycles (v_n=0.19).

In the Tonen system without precracking, the values of normalized shear stress were also higher for the flexure-delamination failure than for the shear-delamination for the same number of freeze-thaw cycles, see Figure 5.3. There was also a consistent decrease in strength with an increase in the number of freeze-thaw cycles. It can be concluded that for this set of specimens an increase in number of freeze-thaw cycles led to a 22% decrease in the shear capacity for 300 F.T. cycles (v_n=0.20).

For the Sika system with precracking, the occurence of vertical shear failure was considered a premature failure that did not allow measuring the effect of the freeze-thaw cycles on the bond strength of the CFRP glued-on system. Considering only the values for shear-delamination and flexure-delamination, it was observed that the decrease in strength was 10% for 300 F.T. cycles whereas a decrease of 20% was observed after 200 freeze-thaw cycles (see Figure 5.4).

For the Tonen system a precracking stage influences the decrease in strength due to the effect of the freeze-thaw cycles. A decrease of the average shear stress of 22% for 300 freeze-thaw cycles was obtained for the non-precracked beams whereas for the precracked beams this value was 39%. However the average value for the normalized shear stress at 300 freeze-thaw cycles for both conditions was very close: v_n=0.20 for precracked beams, and v_n=0.19 without precracking. It can be shown that at 0 F.T. cycles the cracking condition influences the capacity of the beam, whereas after 300 F.T. cycles, the freezing and thawing effect dominates, see Figure 5.5.

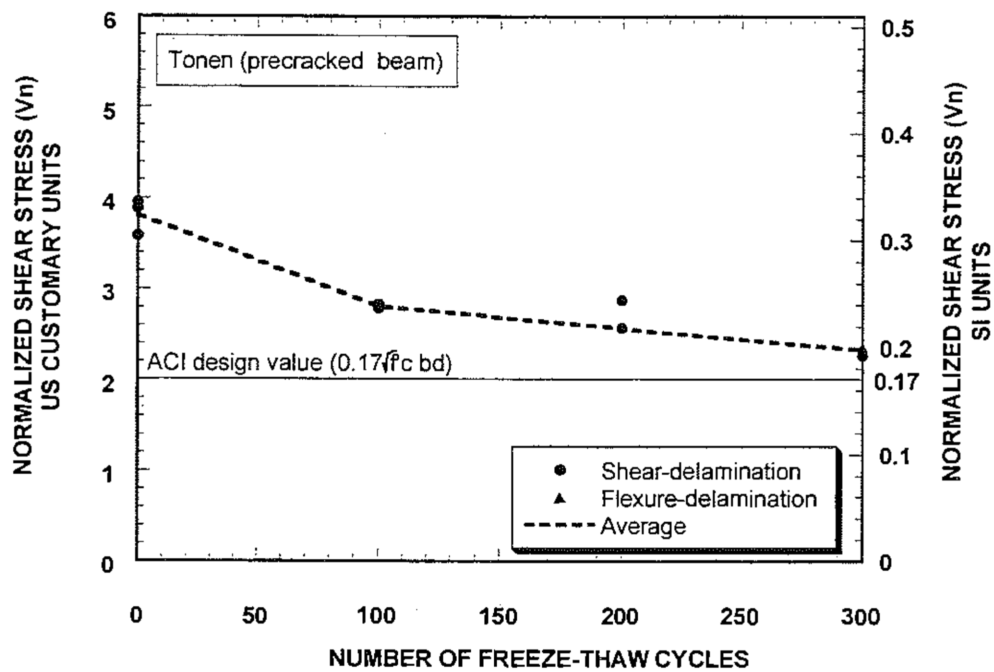


Figure 5.2 Tonen Precracked: Normalized Max. Shear Stress vs Number of F.T. Cycles

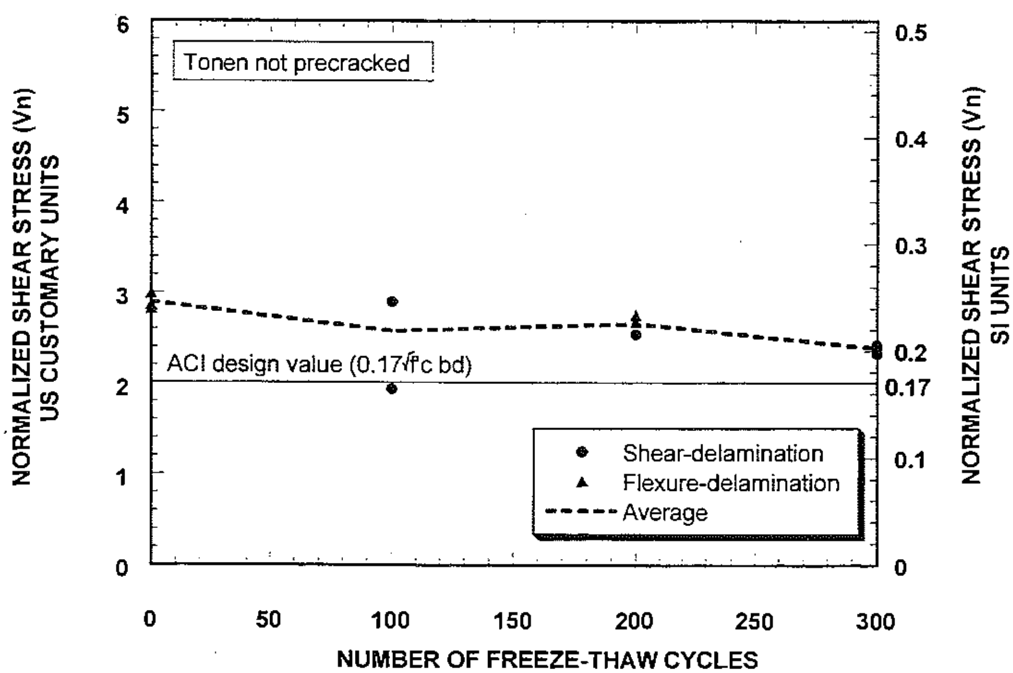


Figure 5.3 Tonen not Precracked: Normalized Max. Shear Stress vs Number of F.T. Cycles

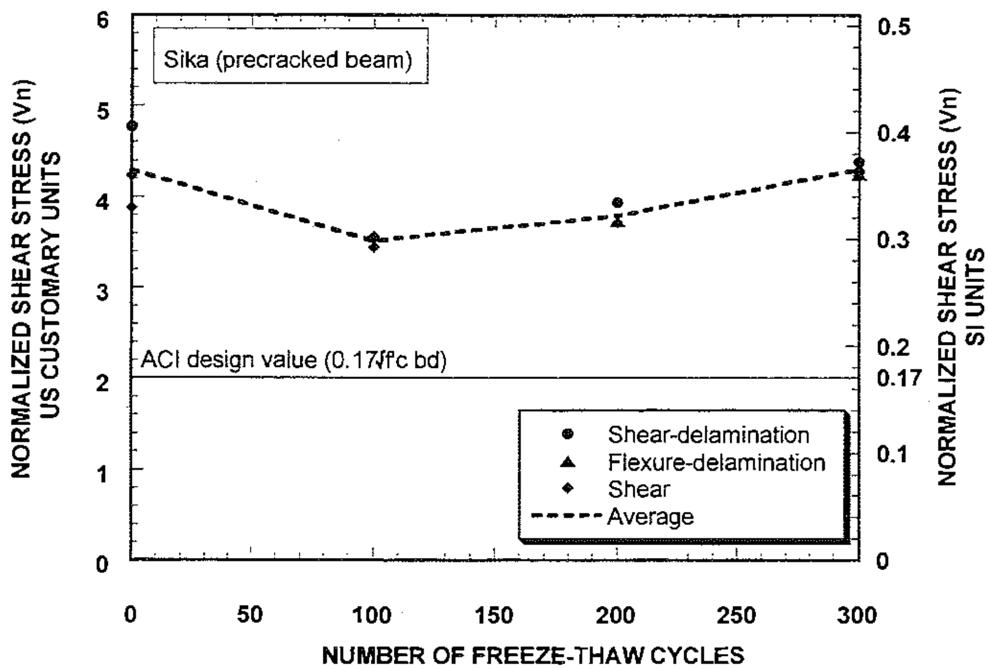


Figure 5.4 Sika Precracked: Normalized Max. Shear Stress vs Number of F.T. Cycles

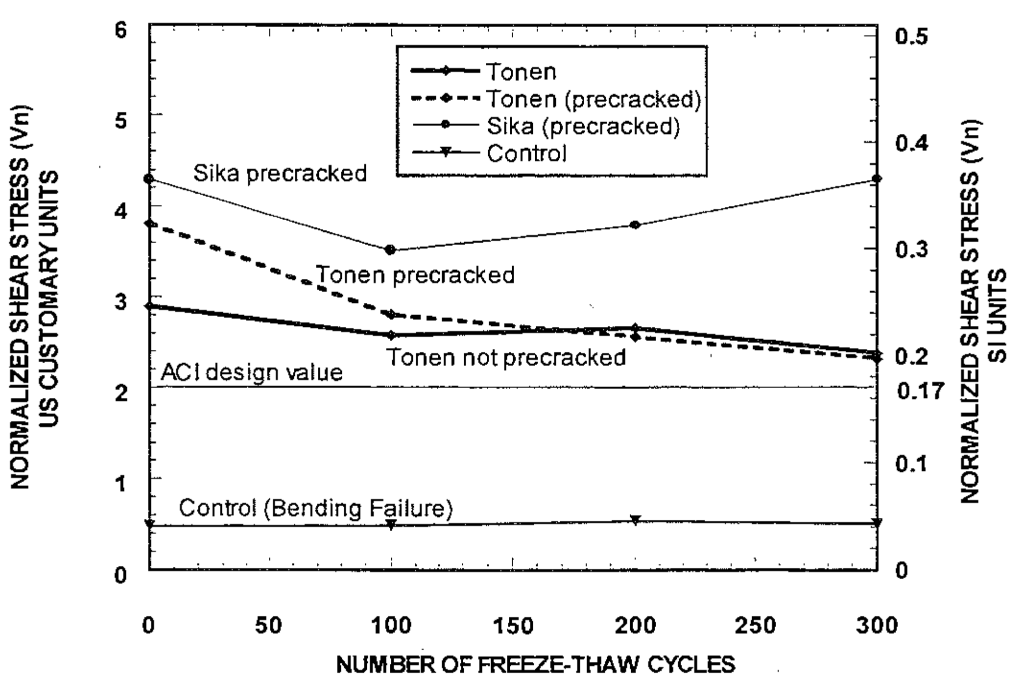


Figure 5.5 Average Normalized Max. Shear Stress at Failure vs Number of F.T. Cycles

For the Sika system, ignoring shear failure, the decrease in strength was less significant but also less consistent. Only a decrease of 10% (v_n=0.36) was observed after 300 freeze-thaw cycles.

5.3. Flexural Strength

Moment-deflection curves were plotted for 0, 100, 200 and 300 freeze-thaw cycles. Figures 5.6 to 5.9 show typical moment-deflection curves for the Tonen and Sika systems as well as the control beams for these numbers of cycles. See appendix C for additional moment-deflection curves for each strengthening system.

A decrease in the moment capacity of the strengthened specimens as well as maximum deflection was observed for an increasing number of freeze-thaw cycles. These findings were analogous to those for shear strength.

The same stiffness was observed for all the specimens (including control specimens) at the pre-yielding stage. At the post yielding stage, the Sika system had a higher stiffness than the Tonen system (for both precracked and non-precracked beams).

For the control specimens no decrease in the moment capacity due to the freeze-thaw cycles was observed. However, as mentioned before, a decrease in the maximum deflection was also observed.

For the Tonen system a decrease in the moment capacity was found for both precracked and non-precracked beams. For the precracked beams, a decrease of the average moment capacity of 27% was found for 100 F.T. cycles, 29% for 200 F.T. cycles and 38% for 300 F.T. For the non-precracked beams the decrease was less drastic (15% was found for 100 F.T. cycles, 19% for 200 F.T. cycles and 20% for 300 F.T).

The maximum deflection was very sensitive to the effect of the freeze-thaw cycles. For precracked beams, a reduction of 15% for the average deflection value was found for 100 F.T. cycles, 40% for 200 F.T. cycles and 43% for 300 F.T. For the non-precracked beams, a value of 3% was found for the average deflection at 100 F.T. cycles, 23% for 200 F.T. cycles and 19% for 300 F.T.

For the Sika system a variation in the moment capacity was also found. A decrease of 11% for the average value of the moment capacity was found for 100 F.T. cycles, 13% for 200 F.T. cycles and 4% for 300 F.T. For average deflection values, a decrease of 15% was found for 100 F.T. cycles, 15% for 200 F.T. cycles and 7% for 300 F.T.

As noted above, for the Tonen and Sika systems an overall decrease in the moment capacity as well as the maximum deflection was observed for an increase in the number of freeze-thaw cycles. The Tonen system presented the higher rate of decrease of moment capacity for the precracked beams (38% for 300 F.T. cycles). As expected the lower rate was found in the Sika system considering that, as indicated before, the predominant failure mode was shear.

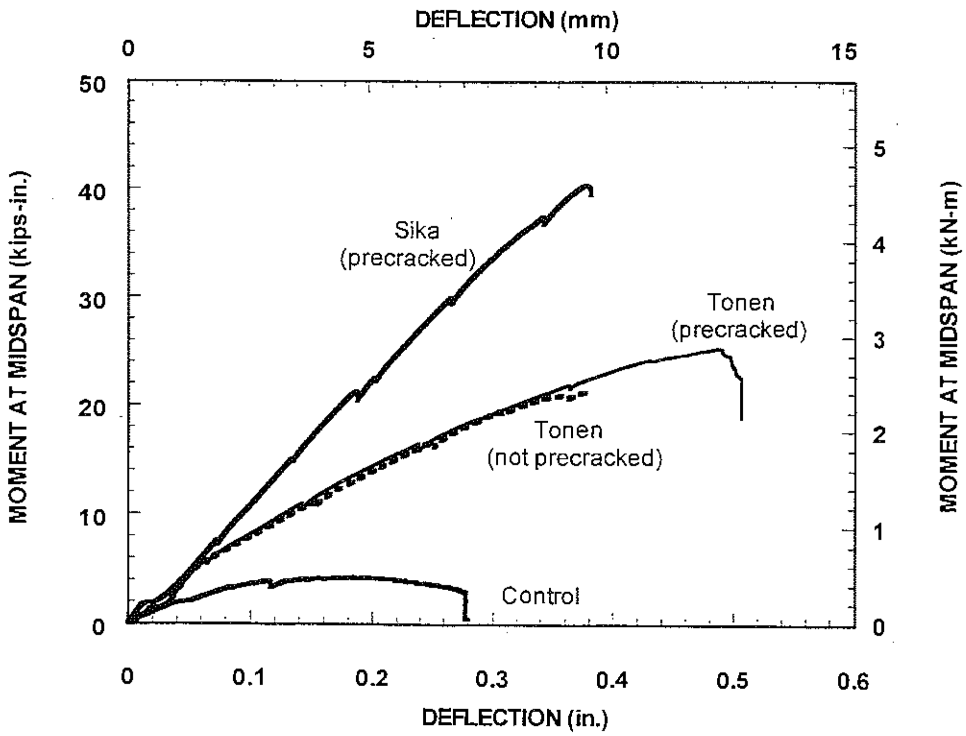


Figure 5.6 Moment-Deflection Curves for 0 F.T. Cycles

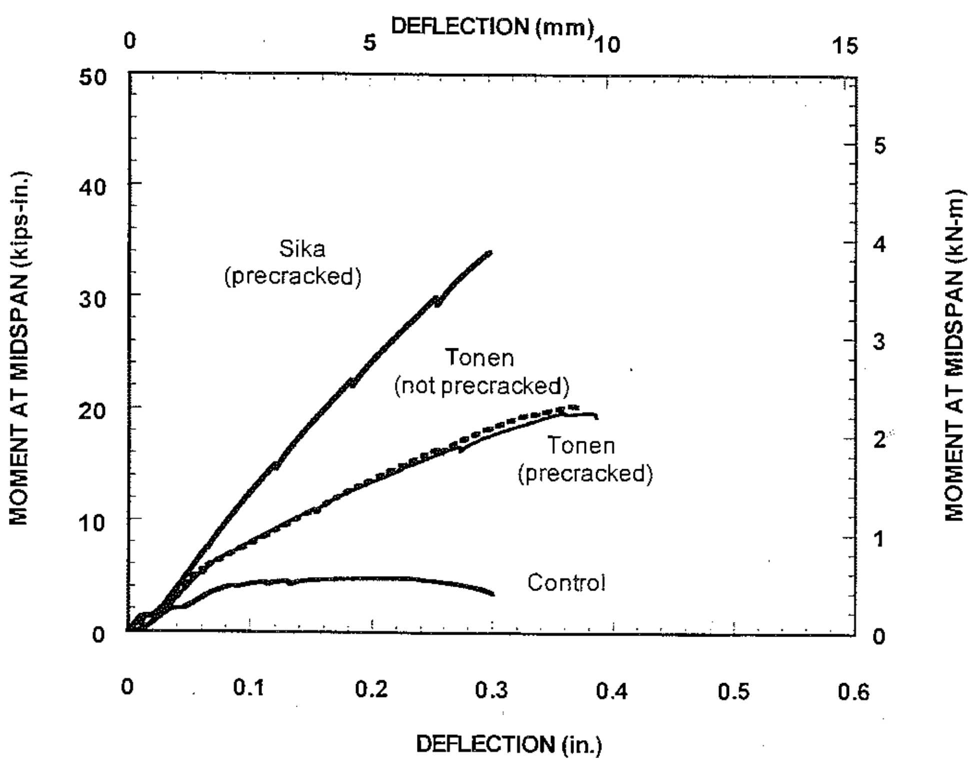


Figure 5.7 Moment-Deflection Curves for 100 F.T. Cycles

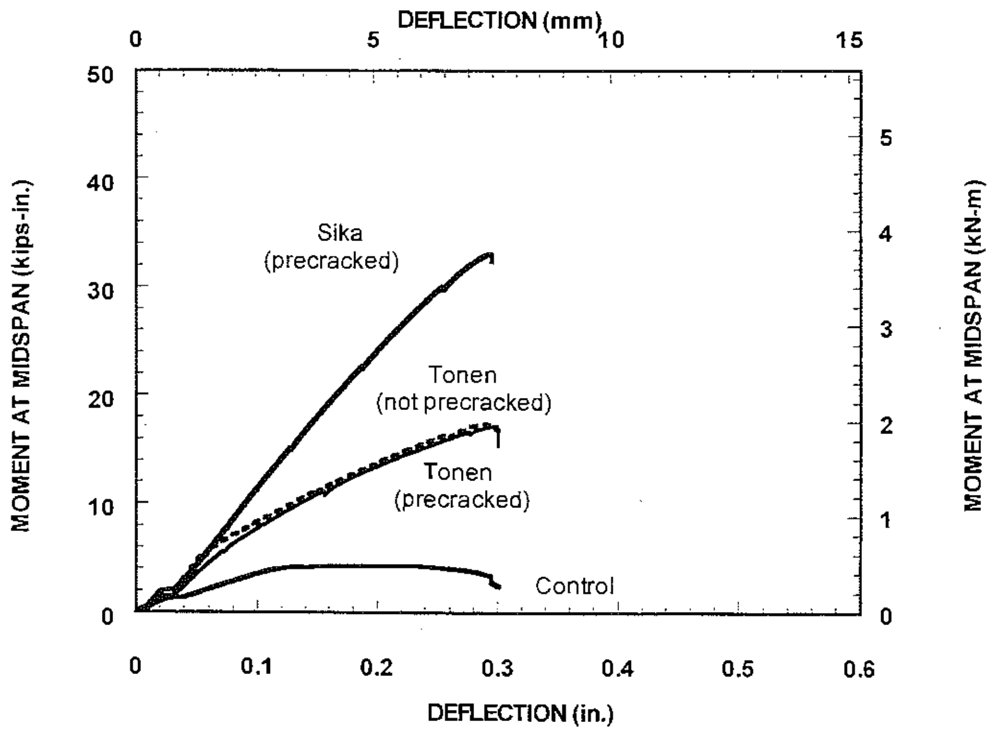


Figure 5.8 Moment-Deflection Curves for 200 F.T. Cycles

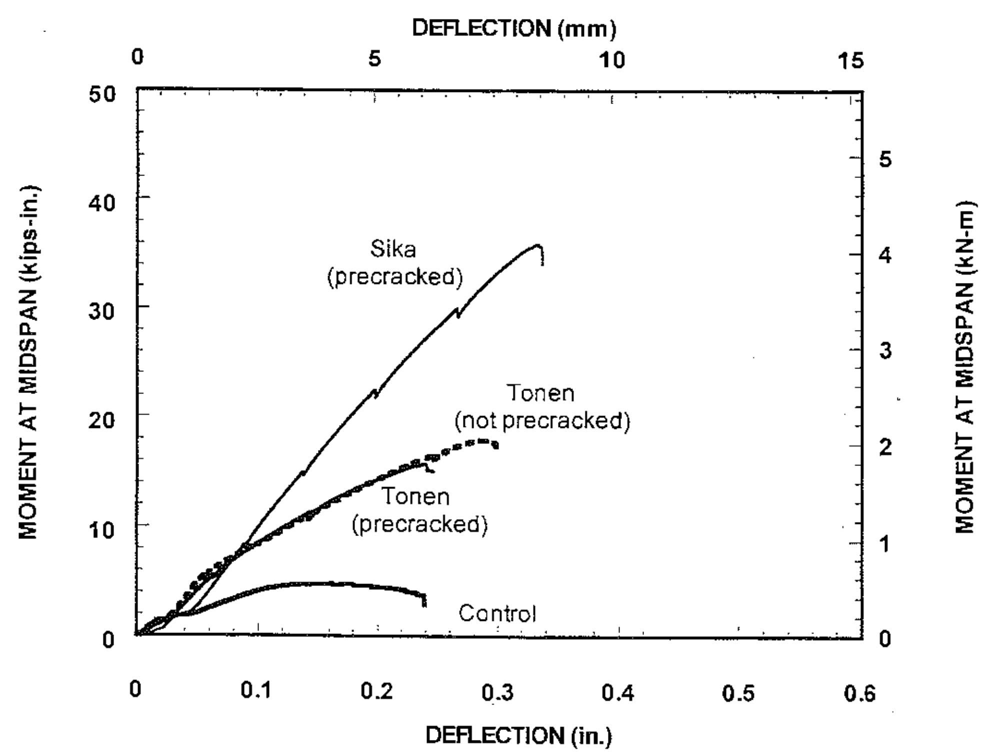


Figure 5.9 Moment-Deflection Curves for 300 F.T. Cycles

Researchers at the University of Bologna [XX-47] found that for their specimens strengthened with rigid CFRP plates (like the Sika system), high shear stress concentration at the end of the plate was responsible for the failure mechanism of the beam.

5.4. Delamination Length

Delamination length was calculated with respect to the total length of the corresponding CFRP laminate. Figures 5.10, 5.11 and 5.12 present the delamination length versus number of freeze-thaw cycles for each one of the strengthening systems.

For all the systems, the delamination length found in specimens that failed either by shear-delamination failure or flexure-delamination was located between the end of the CFRP laminate and the maximum bending area. Confirming the possibility that the experimental observation that the delamination process starts at the tip of a shear or flexural crack

For the Tonen system, a relatively uniform pattern was found for both cases. For the precracked beams, the average delamination length was 23% of the total length for 0 F.T. cycles, 22% for 100 F.T. cycles, 27% for 200 F.T. cycles and 27% for 300 F.T. cycles. For the non-precracked beams, there was no significant variation in the average value of delamination length for each number of freeze-thaw cycles: the average delamination value was 25% for 0 F.T. cycles, 24% for 100 F.T. cycles, 26% for 200 F.T. cycles and 25% for 300 F.T. cycles. For both cases no major differences were found in delamination length for different types of failure modes.

For the Sika system, the variation in average values of delamination length was more significant for all the failure modes. Disregarding the specimens that failed by vertical shear (where the delamination length is considered as 0%), a more uniform pattern could be seen: 29% for 0 F.T., 26% for 200 F.T. cycles and 26% for 300 F.T. cycles. For the Sika system no major differences were found in delamination length for either shear-delamination or flexure-delamination failure.

It can be concluded that the delamination length showed a uniform pattern for both strengthening systems. For the Tonen system, values of delamination length fall within the 22-27% range of the total length. For the Sika system, the range was even more narrow, 26-29%. No influence of either the number of freeze-thaw cycles or the type of failure mode was observed.

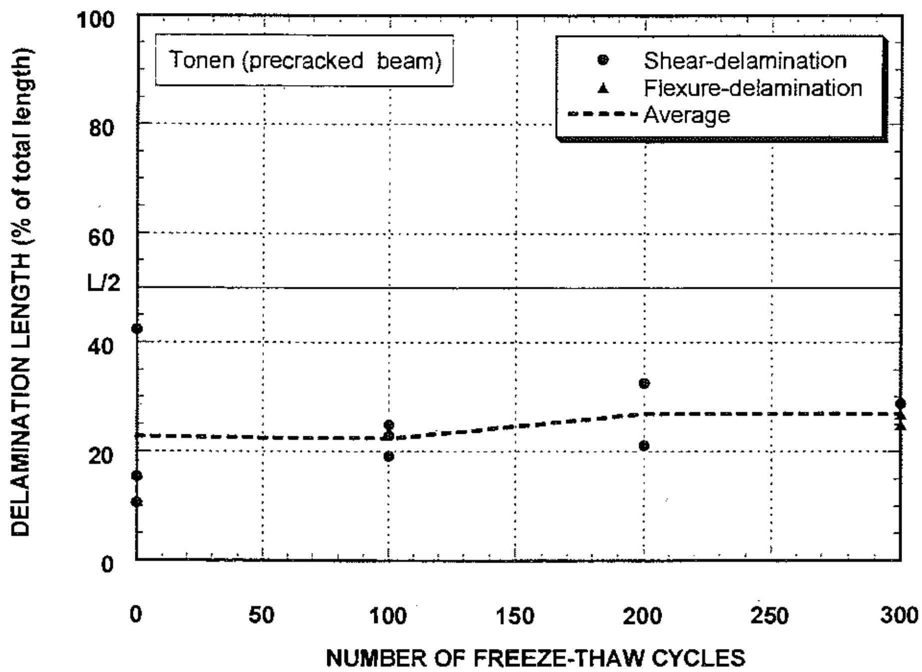


Figure 5.10 Tonen Precracked: Delamination Length vs. Number of F.T. Cycles

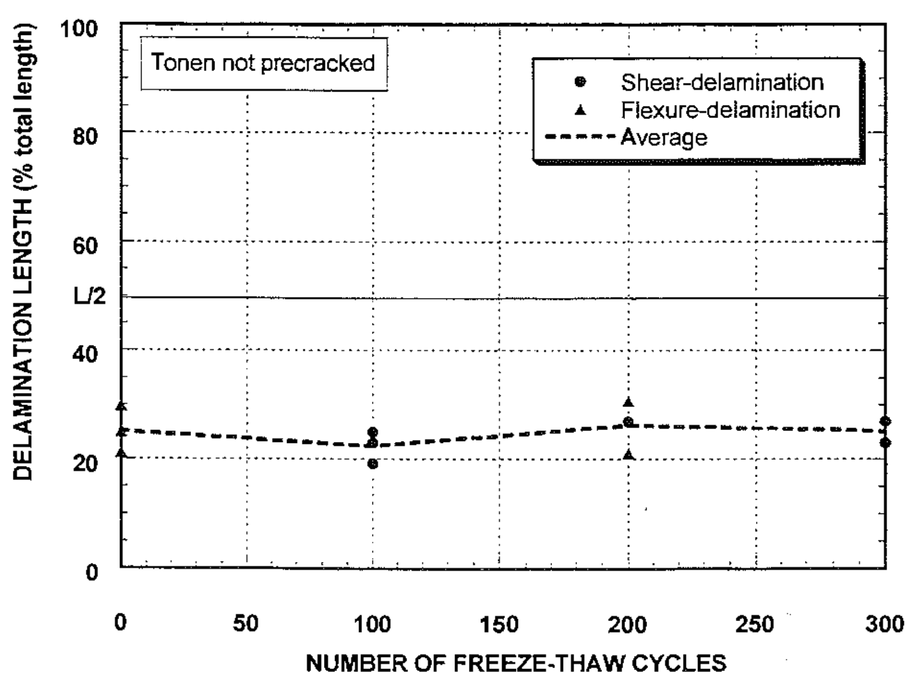


Figure 5.11 Tonen not precracked: Delamination Length vs. Number of F.T. Cycles

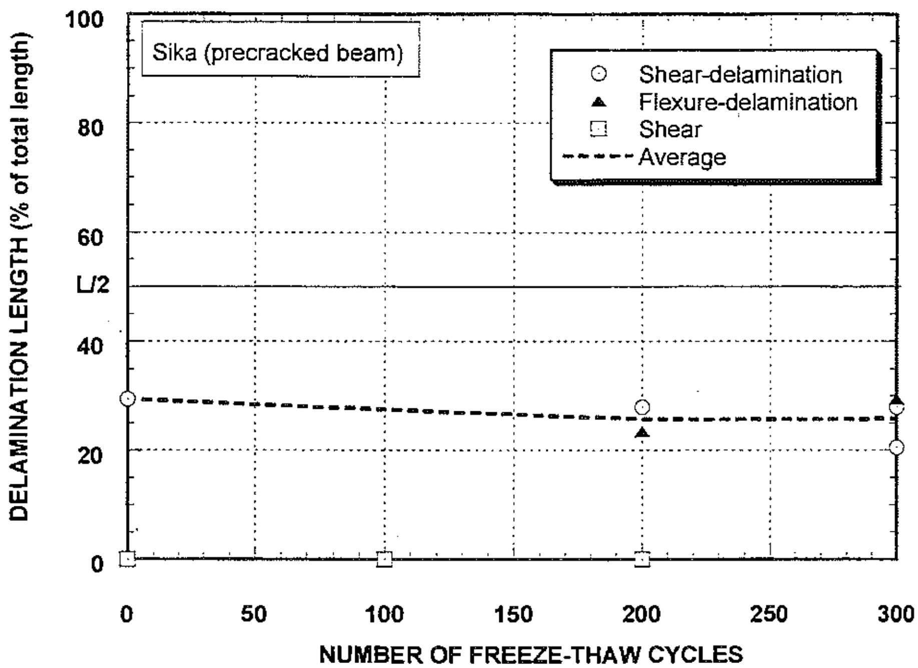


Figure 5.12 Sika Precracked: Delamination Length vs. Number of F.T. Cycles

6. CONCLUSIONS

1. For the control specimens no decrease in the moment capacity or shear strength due to the freeze-thaw (F.T.) cycles was observed. However, a decrease in the maximum deflection was observed.

2. For specimens strengthened with CFRP sheets an overall decrease in the moment capacity as well as the maximum deflection was observed with an increase in number of freeze-thaw cycles. Precracked beams using the Tonen system presented the higher rate of decrease of moment capacity (38% average for 300 F.T. cycles). Non-precracked beams also strengthened with the Tonen system led to an average decrease of 20% for 300 F.T. cycles.

3. The maximum moment capacity of beams strengthened with the Sika system decreased 13% on average for 200 F.T. cycles and 4% average for 300 F.T. cycles. This variation was attributed to the influence of different type of failure modes.

4. The average deflection at maximum load was very sensitive to the effect of the freeze-thaw cycles and the cracking condition. For the Tonen precracked beams a reduction of 43% in deflection was found after 300 F.T. cycles whereas for the Tonen non-precracked beams the decrease was of 19%. Beams using the Sika system showed a smaller rate of decrease in deflection at maximum load (15% average for 200 F.T. cycles and 7% for 300 F.T. cycles)

5. With the Tonen system, the values of normalized shear stress (v_n) for the same number of freeze-thaw cycles were higher for the flexure-delamination failure than for the shear-delamination failure. It was concluded that shear cracks accelerate the interfacial crack propagation.

6. With the Tonen system, precracking the beam influences the decrease in the average normalized shear stress with the freeze-thaw cycles. For 300 freeze-thaw cycles, precracked beams had a decrease of 39% (compared with the strength at zero F.T. cycles) whereas the decrease for the non precracked beams was 22%. However the normalized shear stress after 300 F.T. cycles remained almost the same: $v=0.20\sqrt{f'c}$ for precracked beams, and $v=0.19\sqrt{f'c}$ for non-precracked beams. It can be shown that at zero F.T. cycles the cracking condition influences the capacity of the beam, whereas after 300 F.T. cycles, the freezing and thawing effect dominates.

7. For the Sika system, ignoring vertical shear failure, the decrease in the normalized average shear stress at failure load due to the effect of the freeze-thaw cycles seemed to be less significant than for the Tonen system. A decrease of 10% was observed for 300 freeze-thaw cycles, leading to a value of $0.36\sqrt{f'c}$.

8. For both strengthening systems (Tonen and Sika), the delamination length found in specimens that failed either by shear-delamination or flexure-delamination was located between the end of the CFRP laminate and the maximum bending moment region.

9. The delamination length was quite uniform for both strengthening systems. For the Tonen system, values of delamination length varied between 140 to 180 mm. For the Sika system, the range was even more narrow, 220-250 mm. No influence of either the number of freeze-thaw cycles or the type of failure mode was observed on delamination length.

7. RECOMMENDATIONS

1. Freeze-Thaw (F.T.) cycles influence the behavior of reinforced concrete beams with glued-on Carbon Fiber Reinforced Plastic (CFRP) laminates. According to this study, with the Tonen system a maximum decrease of 29% of flexural capacity could be expected after 200 FT and 38% after 300 F.T. cycles. With the Sika system, the maximum decrease observed was of 13% after 200 F.T. cycles. It should be pointed out that the influence of the Freeze-Thaw cycles may also affects the concrete strength, but its effect cannot be easily observed from testing a reinforced concrete beam in bending; it is possible that this dual effect explains the decrease in strength. Unless some additional tests are carried out, as a first design approximation, it is recommended that a reduction of 40% in horizontal shear strength be taken to account for freeze-thaw exposure.

2. The value of the horizontal interfacial shear strength can be taken conservatively as 0.17 $\sqrt{f'c}$ for both strengthening systems. Preliminary analyses indicate that this value may be close to 1.70 MPa for Tonen system and 2.64 MPa for the Sika system, considering the effect of the different strengthening level provided by the two systems. Further study of the interface bond behavior is needed in order to refine this value.

3. The minimum value of the development length (or anchorage) of the CFRP laminate should be based on the value of 0.17 $\sqrt{f'c}$. This value could be modified by results from further investigations. It is recommended that the bonded length of the CFRP should be as long as possible in order to avoid an interfacial bond failure and to have a more efficient use of the CFRP sheet strength.

4. Since delamination seems to be controlled by the interface bond between the CFRP laminate and the concrete, which is also controlled by the concrete strength, it is strongly recommended to insure very good surface preparation before application of the strengthening system.

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<u>ASTM - American Standard Specifications</u>

ASTM-1. ASTM C666-92. Standard Test method for Resistance of Concrete to Rapid Freezing and Thawing.

9. APPENDIX A

Design of Reinforced Concrete Beams with CFRP

Design of the reinforced concrete beams strengthened with the CFRP sheets was defined prior to the experimental tests. Expected maximum load was necessary to define the load cell to be used for the experimental load set-up. Based on the stresses found from this previous analysis, a value of development length (or anchorage) of the CFRP sheets was also found. In order to assure that failure will be by delamination it was agree to use a CFRP length that will have an anchorage value smaller than the one predicted previously. Calculations of the stresses and strains for the concrete-CFRP section at ultimate as well as development lengths for the two strengthening systems are presented as follows:

For this particular analysis the cross section was considered as a laminate with different layers of material. The upper layer is composed by concrete (not cracked) and steel subjected to compressive stresses. The thickness of this layer corresponds to the depth of the neutral axis of the section. A second layer is made of steel and concrete cracked where only the steel carries tensile stresses. The third layer is composed by the CFRP laminate placed in a longitudinal orientation ($\theta = 0°$) and subjected to tensile stresses due to the bending external moment. Figure 1A shows the cross section to be analyzed by the strain compatibility method.

Stress-Strain Relationship for Steel and Concrete
For this analysis, concrete was considered a material that does not carry any tensile load. The compressive stress-strain relation was based on the model proposed by Hognestad [3]. The ascending branch was modeled with a parabolic function. The descending branch was modeled with a linear function.

e_{cm} = strain of the concrete at any particular point
fc = stress of the concrete corresponding to e_{cm}
f'c = maximum compressive stress (3.45 MPa)
e_o = strain corresponding f'c (assumed 0.002 for this model)
note that e_o is the strain at peak load and that the strain at failure (generally taken by ACI as 0.003) is a result of the analysis.

for $0 \leq e_{cm} \leq e_o$, $\quad fc = f'c\,(2\,(e_{cm}/e_o) - (e_{cm}/e_o)\_)$

for $e_o \leq e_{cm}$, $\quad fc = f'c\,(1\text{-}150\,(e_{cm} - e_o))$

Figure 2A shows the stress- strain relationship based on this model.

The stress-strain relationship of the steel was chosen so it would fit to the experimental curves obtained from the tensile testing of the mesh coupons. A stress-displacement curve of the steel mesh is presented in Figure 3.1. of the main report.

e_s = strain of the steel at a particular point

fs = stress of the steel at e_s
fy = yielding stress of the steel = 400 MPa (from experimental data)
fsu = 482 MPa (from experimental data)
Es = 200 GPa
e_u = strain at failure =0.10 (from experimental data)

for $e_s \leq e_y$, $fs = Es\, e_s$
for $e_y \leq e_s \leq e_m$, $fs = fy + (fsu-fy)\,(2j - j\_)$
where $j = (e_s-e_y)/(e_{sm}- e_y)$ $e_{sm} = 0.031$

Figure 3A show the strain-stress relationship based on this model.

Stress- Strain Relationship for CFRP Laminates
Since all the fibers of the CFRP sheets are unidirectional (in the longitudinal direction), it will be assumed that

$$\sigma_x = E_{CFRP}\, e_{CFRP}$$

where σ_x are the longitudinal stresses at the level of the CFRP and e_{CFRP} is the strain at that level of the CFRP. E_{CFRP} is the modulus of elasticity of the CFRP sheet.

- Tonen system mechanical properties:

Information provided by the supplier:
Modulus of Elasticity E_1 = 227.53 GPa
Tensile strength σ_x = 3.48 GPa
thickness t_f = 0.165 mm
Strain at failure e_u = 1.5%

- Sika system mechanical properties:

Information provided by the supplier:
E_1 = 150 GPa
t_f = 1.2 mm
e_u = 1.4%

Strain Compatibility Method

The strain compatibility method derives from one of the basic assumptions in flexure theory: it is assumed that stresses in the concrete and reinforcement can be computed from the strains using stress-strain curves for concrete and steel [5].

In order to find the moment capacity of the section, a compression strain of 0.003 for the concrete was assumed, based on ACI concrete block failure model.

The location of the neutral axis (kd) was found through an iterative process where equilibrium of internal forces was checked for a defined value of e_{cm}. Once equilibrium was reached, the internal bending moment was calculated. Following this procedure, a moment-curvature curve can be calculated for a particular cross section, see Figure 1A.

Tonen System

$kd = 18$ mm

$e_{cm} = 0.003$ $\quad Cc = 35606$ N

$e_{c1} = 0.00046$ $\quad C1 = 3256$ N

$\sum Ci = 38862$

$e_{s2} = -0.00208$ $\quad Ts2 = 1618$ N

$e_{s3} = -0.00462$ $\quad Ts3 = 1674$ N

$e_{s4} = -0.00771$ $\quad Ts4 = 6958$ N

$e_{CFRP} = -0.00986$ (< 1.5%) $\quad T_{CFRP} = 28426$ N

$\sum Ti = 38676$

$M = 2435$ N-m

$P = M*2/L = 17302$ N

Sika system

$kd = 33$ mm

$e_{cm} = 0.003$ $\quad Cc = 63300$ N

$e_{c1} = 0.0016$ $\quad C1 = 2539$ N

$\sum Ci = 65954$

$e_{c2} = 0.000229$ $\quad C2 = 115$ N

$e_{s3} = -0.00115$ $\quad Ts3 = 1039$ N

$e_{s4} = -0.00277$ $\quad Ts4 = 6530$ N

$e_{CFRP} = -0.00481$ (< 1.4%) $\quad T_{CFRP} = 66390$ N

$\sum Ti = 67273$

$M = 4179$ N-m

$P = M*2/L = 21937$ N

The strains in the CFRP laminated for all the cases are less than the maximum tensile strain (strain at failure).

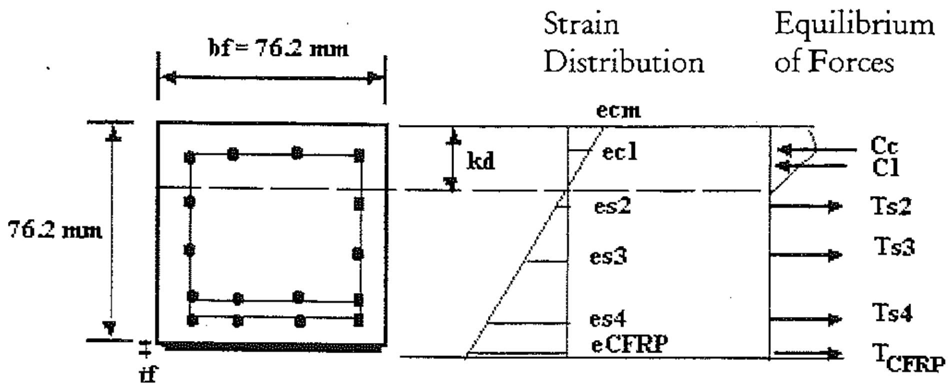

Figure 1A. Cross Section to be Analyzed Using Strain Compatibility Method

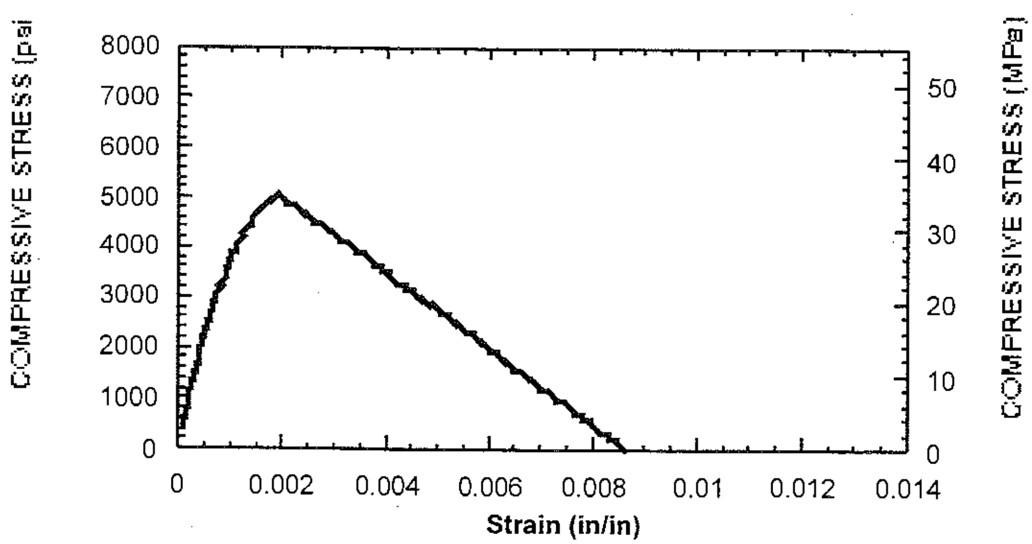

Figure 2A. Stress-Strain Relationship for Concrete in Compression

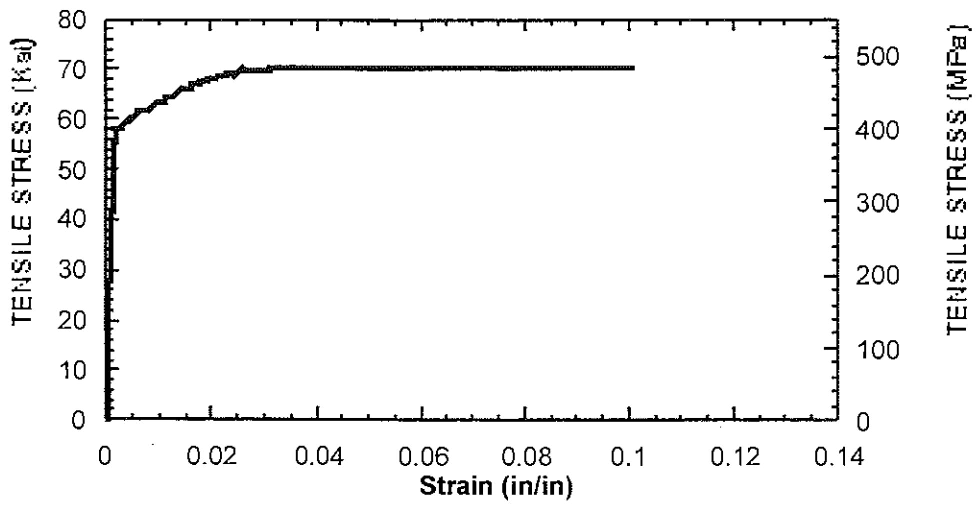

Figure 3A. Stress-Strain Relationship for Steel Mesh

Prediction of the Delamination Length

- Uniform Shear Stress Model

According to the experimental data, it was found that most of the cases of failure of the bond interface between the CFRP sheet and the concrete laminate started at a flexural or shear crack next to one of the point loads and progressed toward the end of the CFRP sheet. Since the shear forces are uniform in the shear span of a four point bending test, an uniform interface shear stress (τ_s) model can be considered [5].

Therefore $l_d = T_{CFRP} / (b^* \tau_s) = (\sigma_{CFRP} * t_f) / \tau_s$

And $\sigma_{CFRP} = E_{CFRP}\, e_{CFRP}$

where l_d (development length) is the shear span for the CFRP laminate and T_{CFRP},σ_{cfrp} are the tensile force and stress of the CFRP sheet at the point under one of the concentrated loads, point A of the Figure4A. The tensile force, stress and strains were found from the equilibrium of the section according to the strain-compatibility method presented in the previous section. t_f is the thickness of the CFRP plate and τ_s is the shear strength of the interface, assume to be equal to the shear strength of the concrete τ_c.

Shear Strength of the concrete $\tau_c = 0.17\sqrt{f'c} = 998$ KPa
(compressive strength of concrete, f'c = 34.48 MPa)

Values of l_d were calculated for every one of the strengthening systems.

Tonen System

$\sigma_{CFRP} = e_{CFRP}\ E_{CFRP} = 0.00986 * 227.53 = 2.24$ GPa

$l_d = 2.26 * 0.165 / 0.998 = 371$ mm

The minimum value of development length required to avoid delamination is 371 mm However because the objective of this project was to measure the influence of the freeze-thaw cycles on the bond strength of this system, it was of our research group interest to actually guaranty this type of failure. Therefore a smaller value than the one calculated previously was adopted (254 mm) as the development length for the CFRP sheet.
Total length of the Tonen CFRP (lcs, see Figure 4A) = 660 mm.

Sika System

$\sigma_{CFRP} = e_{CFRP}\ E_{CFRP} = 0.00481 * 150.7479 = 0.725$ GPa

$l_d = 0.725 * 1.2 / 0.998 = 872$ mm

The minimum value of development length required to avoid delamination is 872 mm.
Following the same line of though presented above, we will use a smaller value for the development length. Since the maximum length of CFRP for the load set-up is 864 mm and this length gives a value of development length is 356 mm (smaller than 872 mm), it is agreed to use this value as the length for the Sika system.

These values of development length are included in the main report (see page 12) and are consider for analysis of the test results.

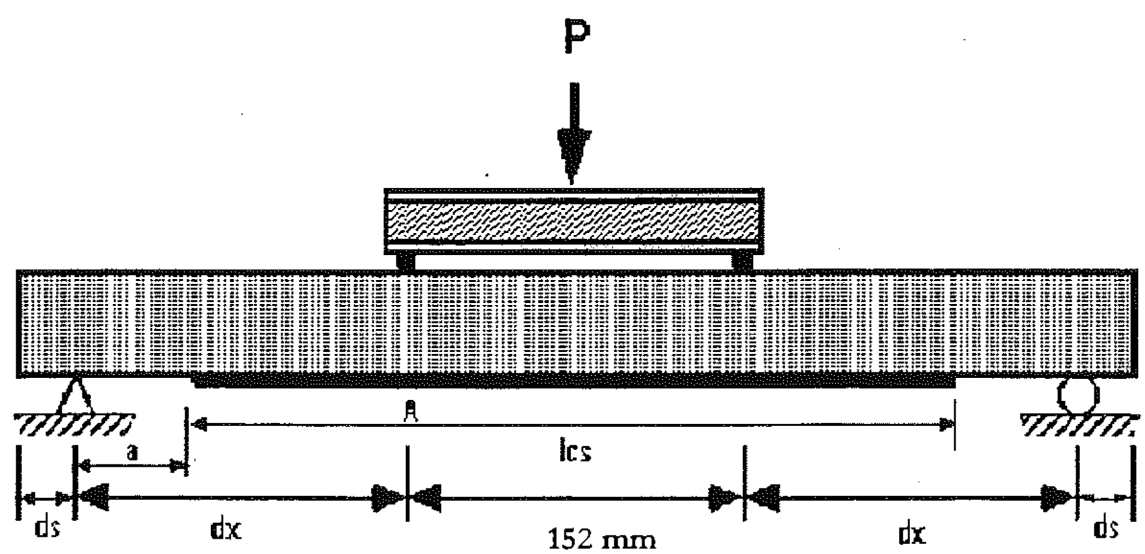

Figure 4A. Flexural Test Load Set-Up

References

[1] Blaschko M., Niedermeier R. and Zielch K. "Bond Failure Modes of Flexural Members Strengthened with FRP." Proceedings of the Second International conference on Composites in Infrastructure. Tucson, Arizona 1998.
[2] Hyer, M. "Stress Analysis of Fiber-Reinforced Composite Materials. WCB/McGraw-Hill publishers. 1998.
[3] Hognestad, E., Hanson, N.W. and McHenry, D., "Concrete Stress Distribution in Ultimate Strength Design," ACI Journal, Vol. 57, No. 2, February 1961
[4] Malek A.M. and Saadatmanesh H. "Design Equations and Guidelines for Reinforced Concrete Beams Strengthened with FRP Plates." Proceedings of the Second International conference on Composites in Infrastructure. Tucson, Arizona 1998.
[5] Wang, C.Y. and Ling F.S. "Prediction Model for the Debonding Failure of Cracked RC Beams with Externally Bonded FRP Sheets." Proceedings of the Second International conference on Composites in Infrastructure. Tucson, Arizona 1998.
[6] Wu Z., Matsuzaki T. and Tanabe K., "Interface Crack propagation in FRP Strengthened Concrete Structures". Proceedings of the Third International Symposium of Non-Metallic (FRP) Reinforcement for Concrete Structures. Vol. 1, Oct. 1997.

10. APPENDIX B

Interface Bond Modeling of Reinforced Concrete Beams Externally Reinforced with Glued-on Carbon Fiber Reinforced Plastic (CFRP) Sheets

The objective of this analysis is to predict the value of the interface shear strength between the CFRP laminate and the concrete when interface bond failure occurred. In order to avoid the influence of freeze-thaw cycles, which will be analyzed in the main report, only specimens at 0 F.T. cycles will be considered. Additionally, only specimens that failed by delamination will be considered.

Table 1B. Maximum External Moment and Shear Forces at 0 F.T. cycles

Strengthening System	Average Max. Shear Force (N)	Average Max. Moment (N-m)
Tonen (not precracked beam)	8669	2422
Tonen (precracked beam)	10062	2810
Sika (precracked beam)	11996	4603

From the failure loads, average values of moment capacity for the section A (see Figure 1B) under a point load were calculated and are presented in Table 1B.
Stresses and strains based on the strain compatibility method and equilibrium of forces were found for this section. The resulting moment was intended to match the values of external moment obtained from the experimental data (see Table 1B).
According to the experimental data, it was found that most of the cases of failure of the bond interface between the CFRP sheet and the concrete laminate started at a flexural or shear crack next to one of the point loads and progressed toward the end of the CFRP sheet. Stress-strain relationships for concrete, steel and the CFRP sheets were the same used in Appendix A.
Notation was also kept consistent with the one used on Appendix A. (See Figure 2B, Appendix B). Note that in the following results, the strain in the concrete top fiber can be larger than 0.003 and is a result of the analysis.

<u>Tonen Not Precracked</u>

kd = 19 mm

e_{cm} = 0.0036 Cc = 37984 N

e_{c1} = 0.00288 C1 = 3272 N $\sum$ Ci = 41300

e_{s2} = 0.00218 Ts2 = 1620 N

e_{s3} = 0.00506 Ts3 = 1683 N

e_{s4} = 0.00795 Ts4 = 6958 N $\sum$ Ti = 42000

e_{CFRP} = 0.01103 (< 1.5%) T_{CFRP} = 31777 N

M = 2503 N-m

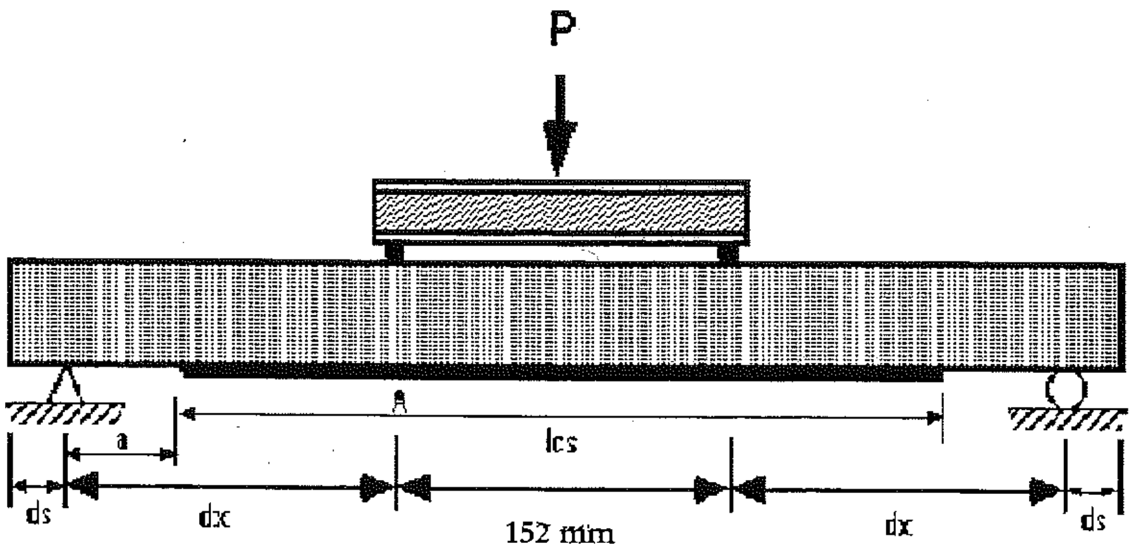

Figure 1B. Cross Section and Load Set-Up

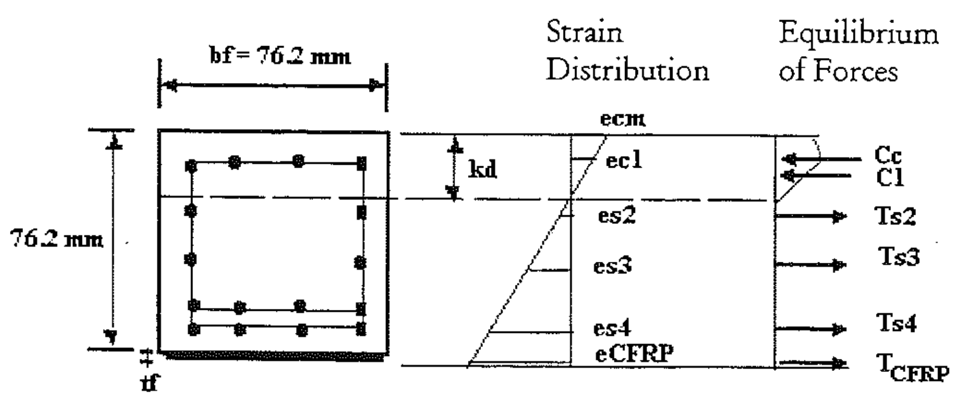

Figure 2B. Cross Section to be Analyzed Using Strain Compatibility Method

<u>Tonen Precracked</u>

kd = 22 mm

e_{cm} = 0.005 Cc = 42305 N

e_{c1} = 0.00346 C1 = 3298 N $\sum$ Ci = 45603

e_{s2} = 0.00193 Ts2 = 1560 N

e_{s3} = 0.00539 Ts3 = 1690 N

e_{s4} = 0.00885 Ts4 = 6781 N $\sum$ Ti = 46000

e_{CFRP} = 0.0125 (< 1.5%) T_{CFRP} = 36012 N

M = 2802 N-m

<u>Sika System</u>

kd = 35.5 mm

e_{cm} = 0.0040 Cc = 70690 N

e_{c1} = 0.00228 C1 = 3245 N $\sum$ Ci = 74392

e_{c2} = 0.000566 C2 = 457 N

e_{s3} = 0.00115 Ts3 = 929 N

e_{s4} = 0.00287 Ts4 = 6544 N $\sum$ Ti = 73863

e_{CFRP} = 0.00481 (< 1.4%) T_{CFRP} = 66390

M = 4459 N-m

Since no tensile failure of the CFRP was observed, it was expected that the strains in the CFRP sheets for all cases were less than in the maximum tensile strain (strain at failure). The values obtained from this analysis corroborate this statement.

It should also be noted that the difference between the calculated and the experimental value of moment is due to the nature of the numerical iteration process. In all cases this difference was less than 3%.

Prediction of the Bond Strength

- **Uniform Shear Stress Model**

According to the experimental data, it was found that most of the cases of failure of the bond interface between the CFRP sheet and the concrete laminate started at a flexural or shear crack next to one of the point loads and progressed toward the end of the CFRP sheet. As shown in Appendix A, an uniform interface shear stress (τs) along the shear span was assumed as a first shear stress model.

Therefore $\tau s = (\sigma_{CFRP} * t_f) / (dx-a)$

where (dx-a) is the shear span for the CFRP sheet (see Figure 1B), t_f is the thickness of the CFRP sheet and σ_{CFRP} is the tensile stress of the CFRP sheet at the point under on of the concentrated loads (point A in Figure 1B).

Values of τs were calculated from every one of the final iterations presented above.

Tonen Not Precracked

$\sigma_{CFRP} = e_{CFRP} E_{CFRP} = 0.011 * 227.53 = 2.50$ GPa

$\tau_s = 2.50 * 0.165 / 254 = 1.63$ MPa ($\approx 0.28 \sqrt{f'c}$)

From the experimental data it was observed that the interfacial debonding process occurred at the level of the concrete surface. ACI suggested a value of $0.17\sqrt{f'c}$ (MPa) for the shear strength of the concrete (τ_c). It is consider that this value could be used as a control to avoid the interface bond failure.

Tonen Precracked

$\sigma_{CFRP} = e_{CFRP} E_{CFRP} = 0.0125 * 227.53 = 2.84$ GPa

$\tau_s = 2.84 * 0.165 / 254 = 1.85$ MPa ($\approx 0.31 \sqrt{f'c}$)

As for the case of Tonen not precracked, the shear strength of the concrete controlled the interface bond failure. The values of τ_s for both cases (precracked and not precracked) were larger than $0.17\sqrt{f'c}$.

Sika system (Precracked)

$\sigma_{CFRP} = e_{CFRP} E_{CFRP} = 0.00481 * 150.7479 = 7.25$ GPa

$\tau_s = 7.25 * 1.2 / 355.6 = 2.44$ MPa ($\approx 0.43 \sqrt{f'c}$)

As for the Tonen system, it was found that the failure occurred at the level of the concrete surface.

- **Non Uniform Shear Stress Model**

For this model a strip of beam in the shear span of length dx was analyzed. From the equilibrium of axial forces a relation was found to estimate the interface CFRP-concrete shear stress value τ_s (see Figure 3B). The value of τ_s is assumed to be uniform along the length dx:

$$\frac{dT_{CFRP}}{dx} - \tau_s \cdot b_f = 0 \qquad (1)$$

Values of τ_s are obtained from solving (1). Since the variation of the compressive forces and tensile forces along the shear span are related to the nonlinearity of the stress-strain relationship of the concrete and steel, the evaluation of τ_s will be done numerically.

Therefore, for a strip of length dx, $$\tau = \frac{\frac{dT_{CFRP}}{dx}}{b_f} \qquad (1)$$

Tonen Not Precracked

Defining x = 0 at point A (see Figure 1). Equilibrium of forces at this point was set using the strain compatibility method.

At x = 0 $\sum C = 41256, \sum T = 10261, \sum T_{CFRP} = 31777$

For dx=18 mm, M=2435 N-m $\sum C = 38862, \sum T = 10190, \sum T_{CFRP} = 28426$

from (1), $\tau = (186167) / 0.0762 = 2.44$ MPa

Tonen Precracked

At x = 0 $\sum C = 45603, \sum T = 10031, \sum T_{CFRP} = 36012$

For dx=20 mm, M=2601 N-m $\sum C = 41256, \sum T = 10261, \sum T_{CFRP} = 31777$

from (1), $\tau = (211750) / 0.0762 = 2.78$ MPa

Sika system

At x = 0 $\sum C = 70690, \sum T = 7473, \sum T_{CFRP} = 66390$

For dx=24 mm, M=4179 N-m $\sum C = 65954, \sum T = 7569, \sum T_{CFRP} = 59704$

from (1), $\tau = (278583) / 0.0762 = 3.66$ MPa

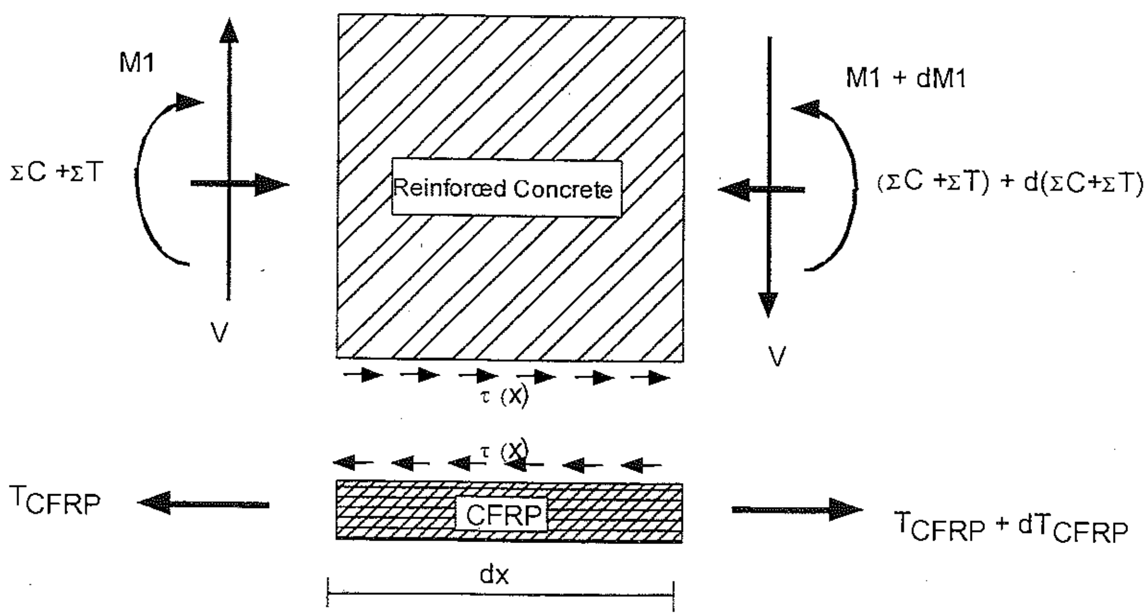

Figure 3B. Free Body Diagram

- **Comparison of Results**

Table 2B presents a comparison of results of τ_s values obtained from the uniform shear stress model and the non uniform shear stress model. The third column represents the calculation of the average vertical shear stress $v_1 = V/(b_f*d)$, where V = shear force(from Table 1B), b_f = width of concrete beam = 76.2 mm and d = distance from maximum compressive strain to centroid of steel at tension = 63.5 mm. The fourth column represents the calculation of the $v_2=V/(b*h)$, where h = height of the concrete beam = 76.2 mm.

As can be seen from Figure 4B, model 1 represents the lower bound and model 2 the upper bound of the values of the interface shear strength. Comparing these values with the calculation of v_1 and v_2, we can conclude that the calculation of the average vertical shear stress is an indirect measure of the value of the interface shear for the Tonen system. Values of v_2 represent the lowest bound (compared to model 1 and 2) and are therefore on the safe side (for a defined value of τ_s).

Values of τ_s for the Sika system were higher than the ones for Tonen system. Recalling that for the Sika system at 0 freeze-thaw cycles, 2 out of 3 beams failed by vertical shear, it is possible that the values of τ_s found are only approximate. Additionally, v_1 and v_2 values calculated were higher than the τ_s values obtained from model 1 and 2. However, v_2 was closer to the τ_s values than v_1. Considering the shear strength of the concrete as $0.17\sqrt{f'c}$ = 0.998 MPa we can conclude that this value can be taken conservatively as the interfacial bond strength.

Table 2B. τ_s values obtained from analysis.

System	Model1 (uniform τ_s) MPa	Model2 (discontinuous τ_s) MPa	v_1=V/bd (MPa)	v_2= V/bh (MPa)
Tonen not precracked	1.63	2.44	1.79	1.48
Tonen precracked	1.85	2.78	2.08	1.73
Sika precracked	2.44	3.66	4.99	4.14

It can be concluded that for an accurate prediction of the bond strength at the interface either model 1 or 2 should be analyzed for all the experimental data obtained. However the values of V/bh obtained follow the same trend as those of either Model 1 or 2, and are recommended as a first approximation in design.

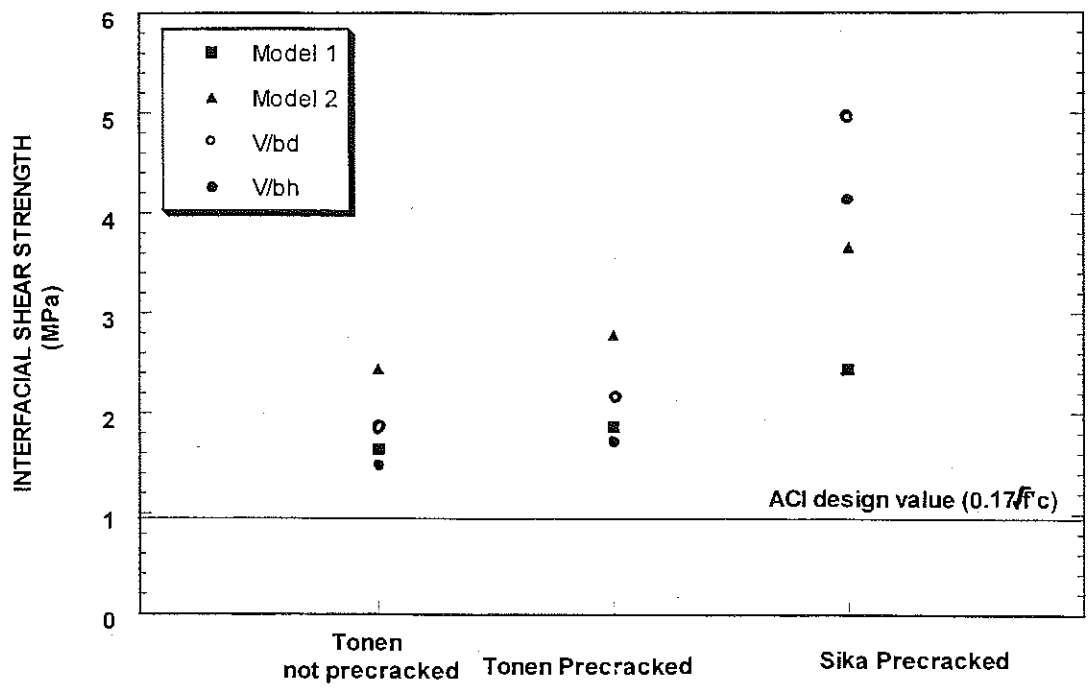

Figure 4B. Interfacial Shear Strength Values

References

[1] Blaschko M., Niedermeier R. and Zielch K. "Bond Failure Modes of Flexural Members Strengthened with FRP." Proceedings of the Second International conference on Composites in Infrastructure. Tucson, Arizona 1998.

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[6] Wu Z., Matsuzaki T. and Tanabe K., "Interface Crack propagation in FRP Strengthened Concrete Structures". Proceedings of the Third International Symposium of Non-Metallic (FRP) Reinforcement for concrete Structures. Vol 1, Oct., 1997.

APPENDIX C

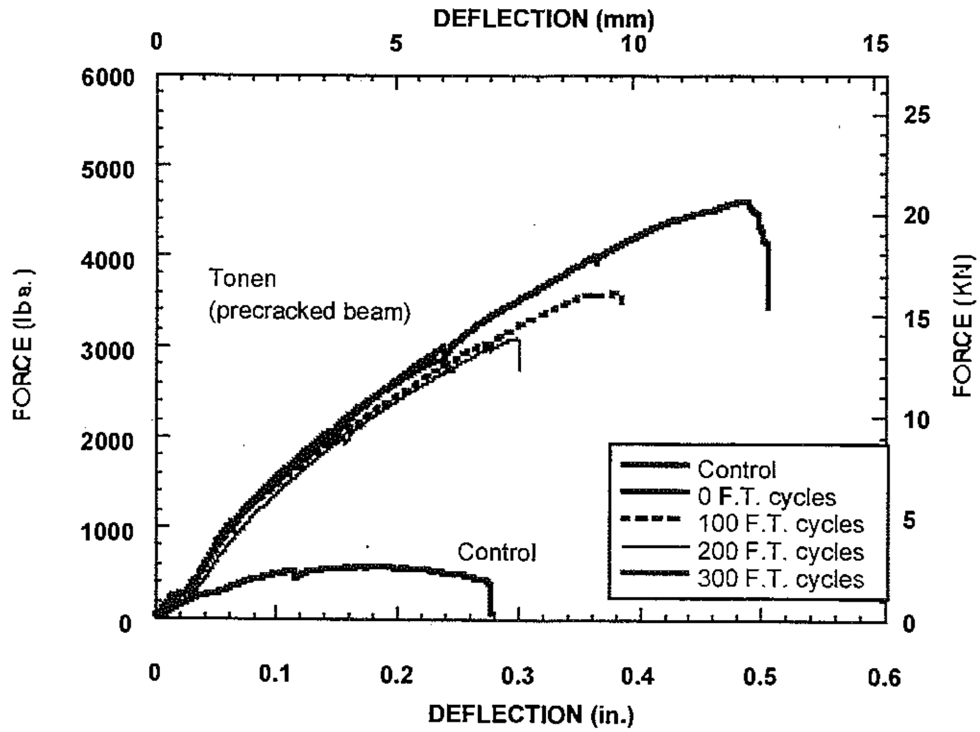

Figure 1C Load-Deflection Curves for Tonen Precracked Beams

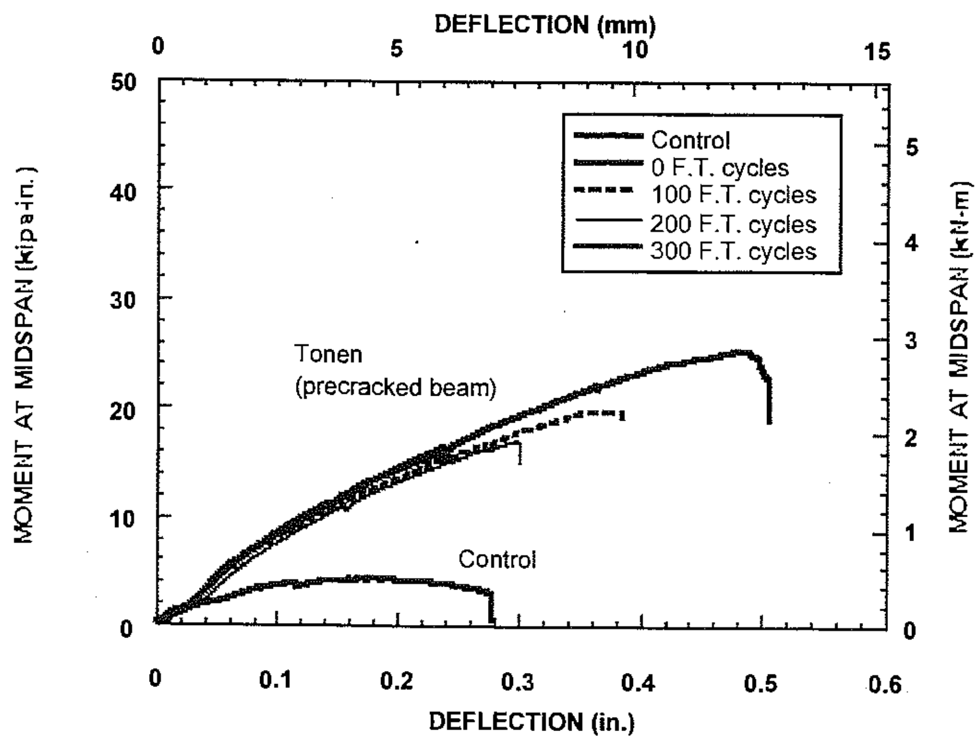

Figure 2C Moment-Deflection Curves for Tonen Precracked Beams

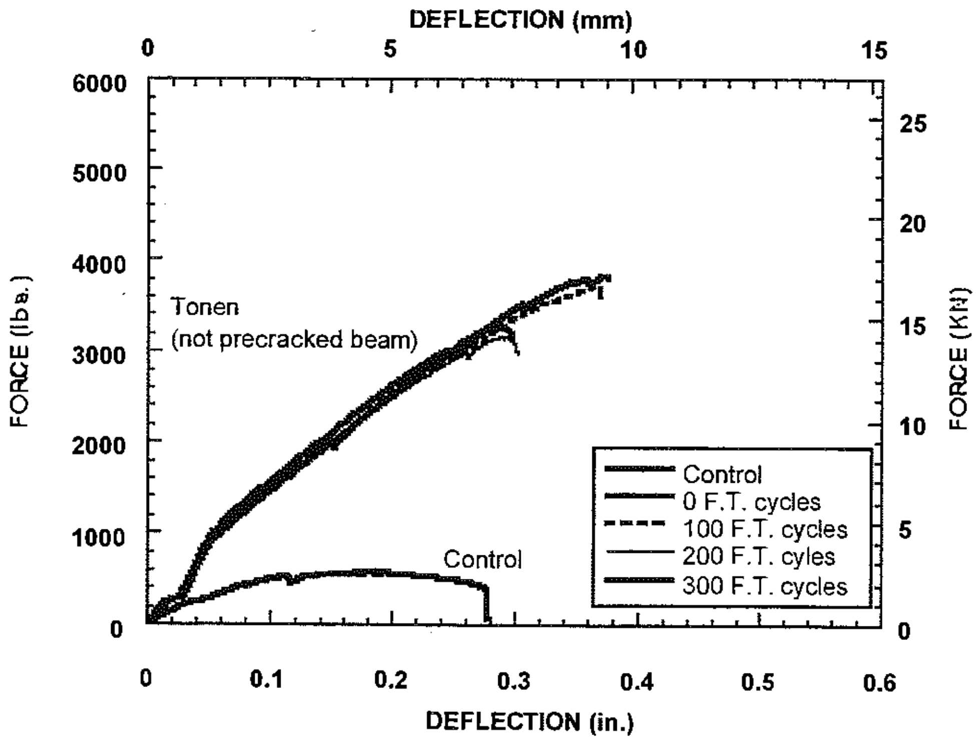

Figure 3C Load-Deflection Curves for Tonen Not Precracked Beams

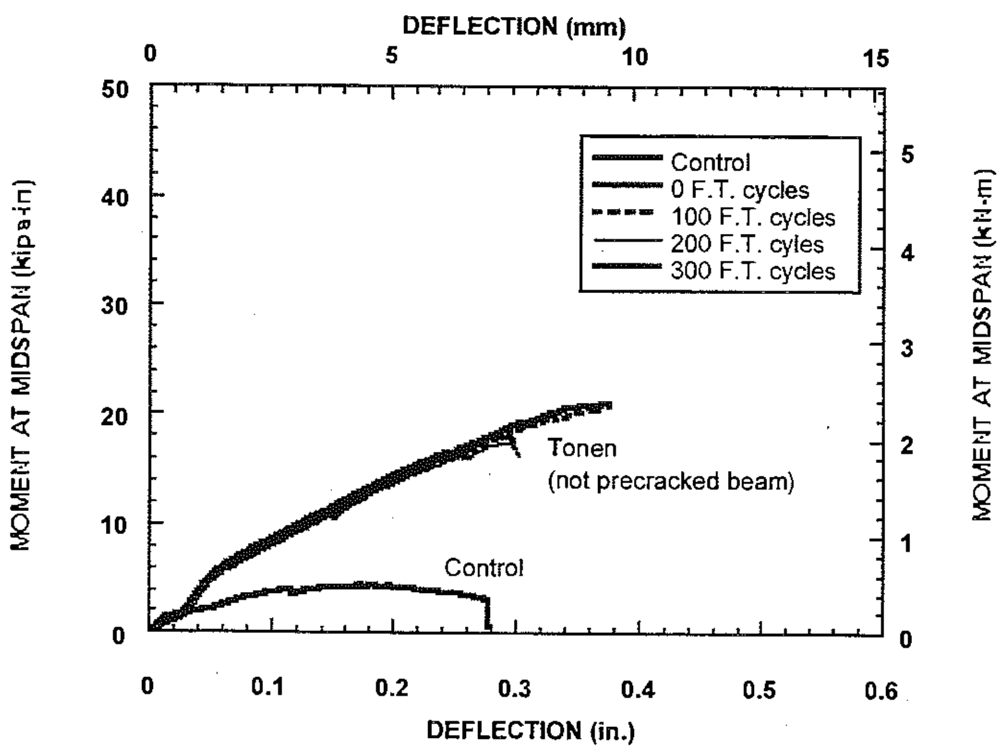

Figure 4C Moment-Deflection Curves for Tonen Not Precracked Beams

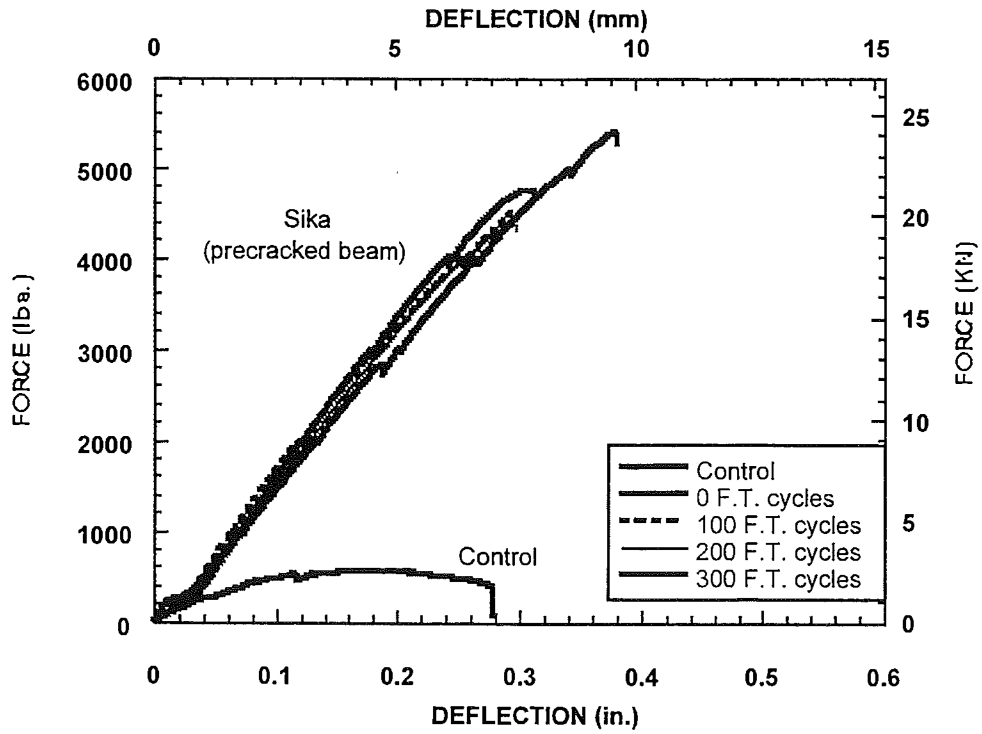

Figure 5C Load-Deflection Curves for Sika Precracked Beams

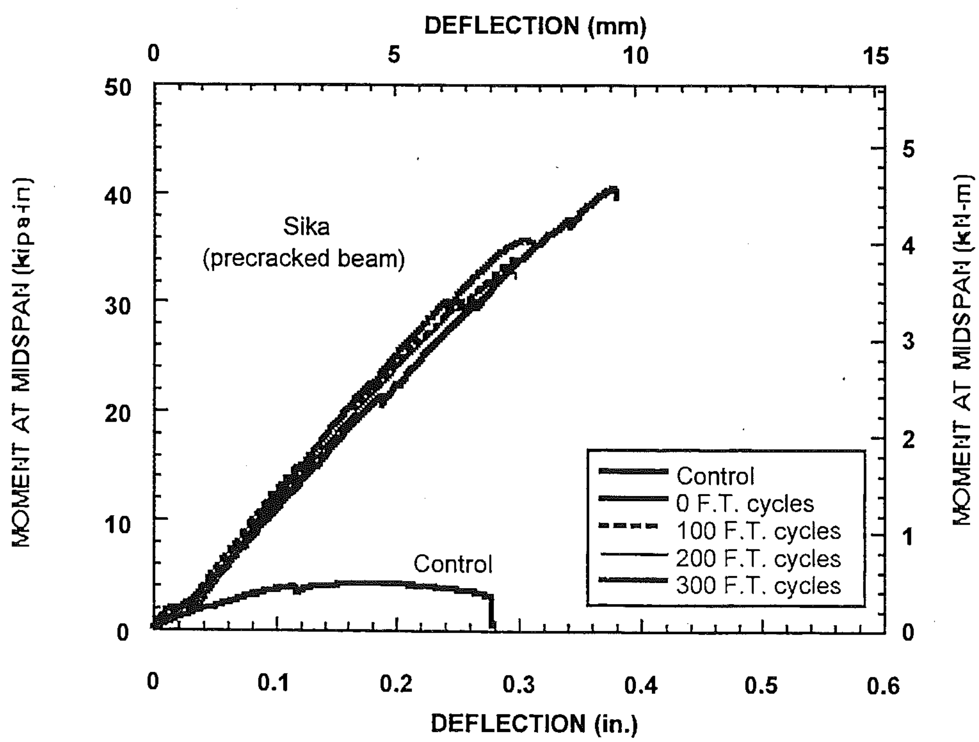

Figure 2C Moment-Deflection Curves for Sika Precracked Beams

APPENDIX D

1. MATERIALS

1.1 STEEL

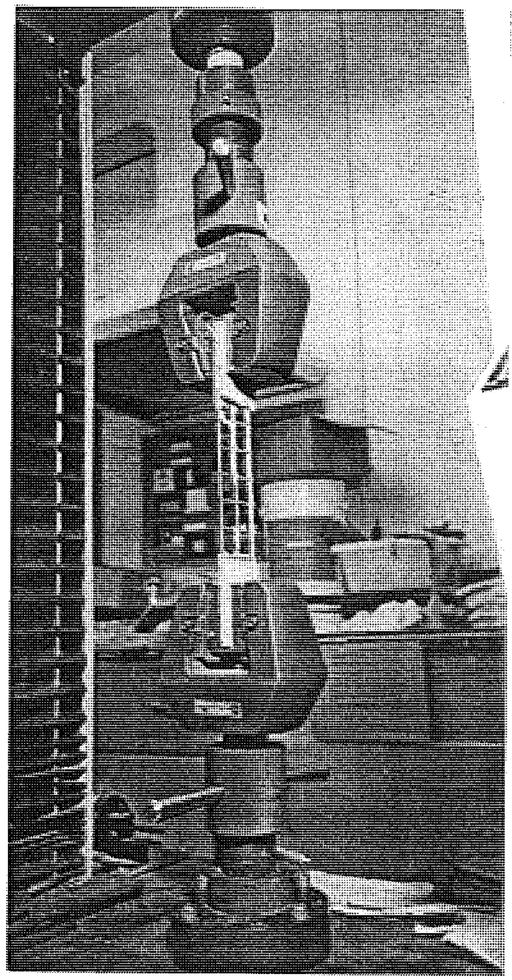

Figure 1. Tensile Test of Steel Mesh Coupons

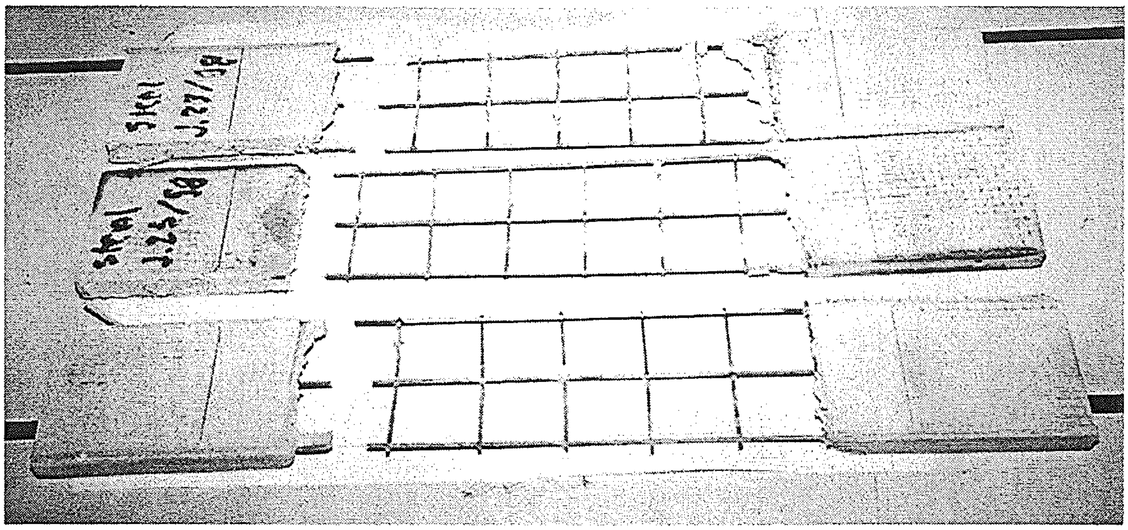

Figure 2. Steel Mesh Coupons

2. TEST SET-UP

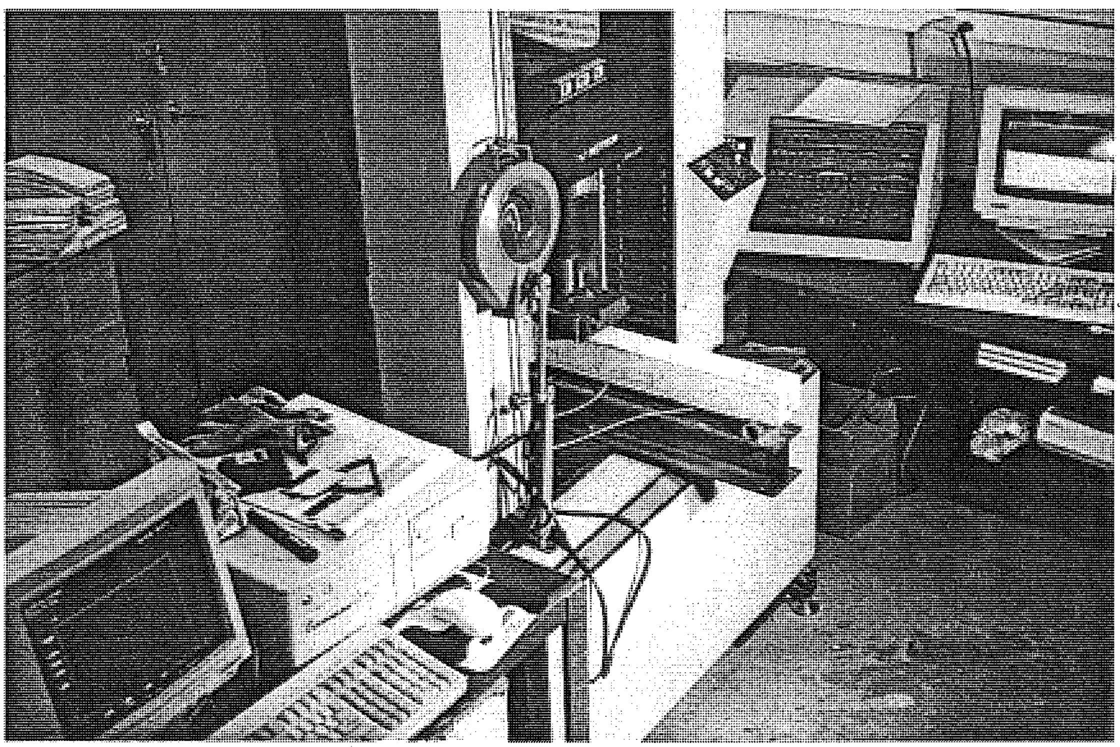

Figure 3. Test Set-Up

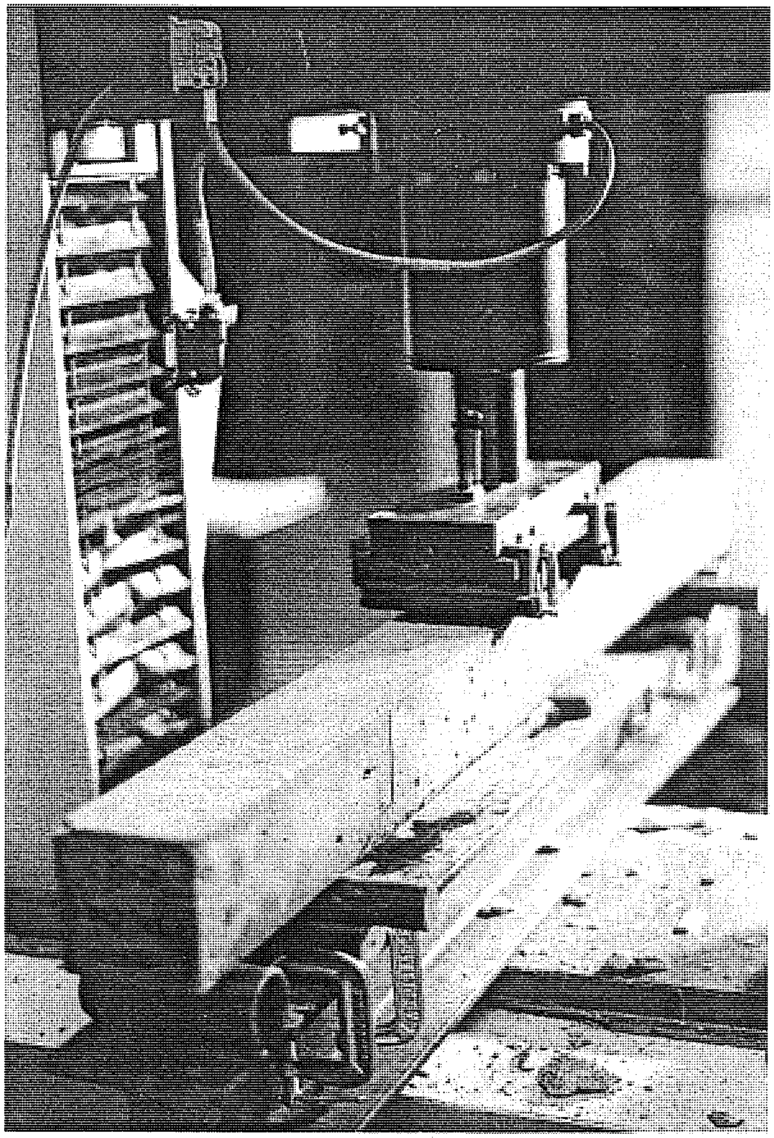

Figure 4. Bending Test of F.T. Specimens

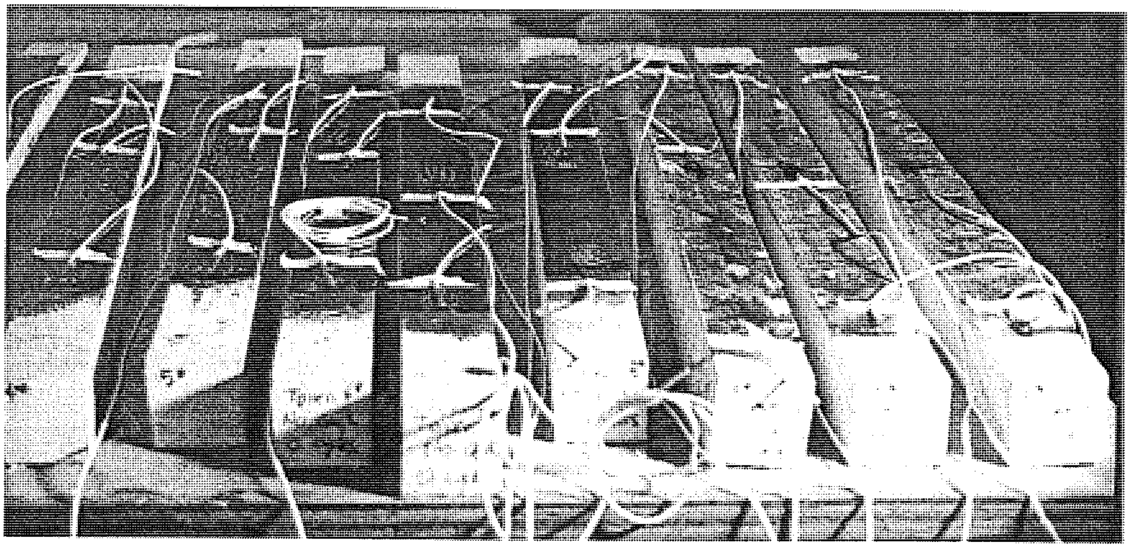

Figure 5. Instrumentation of Specimens at 0 F.T. Cycles

3. FAILURE MODES

3.1 Control Specimens (Yielding of the steel reinforcement, flexural cracks)

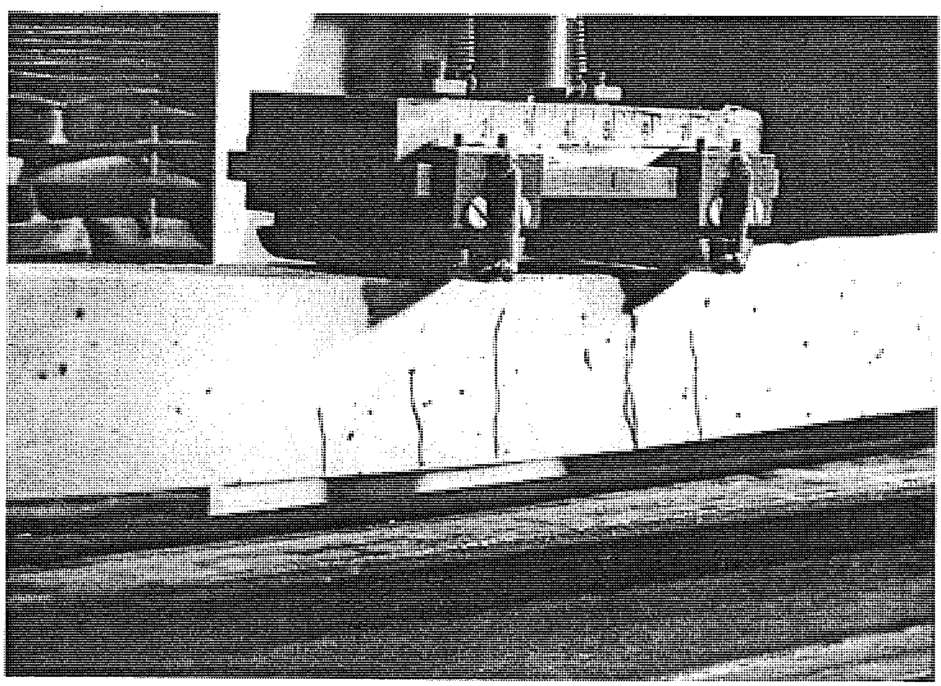

Figure 6. Bending Test of Control Specimens

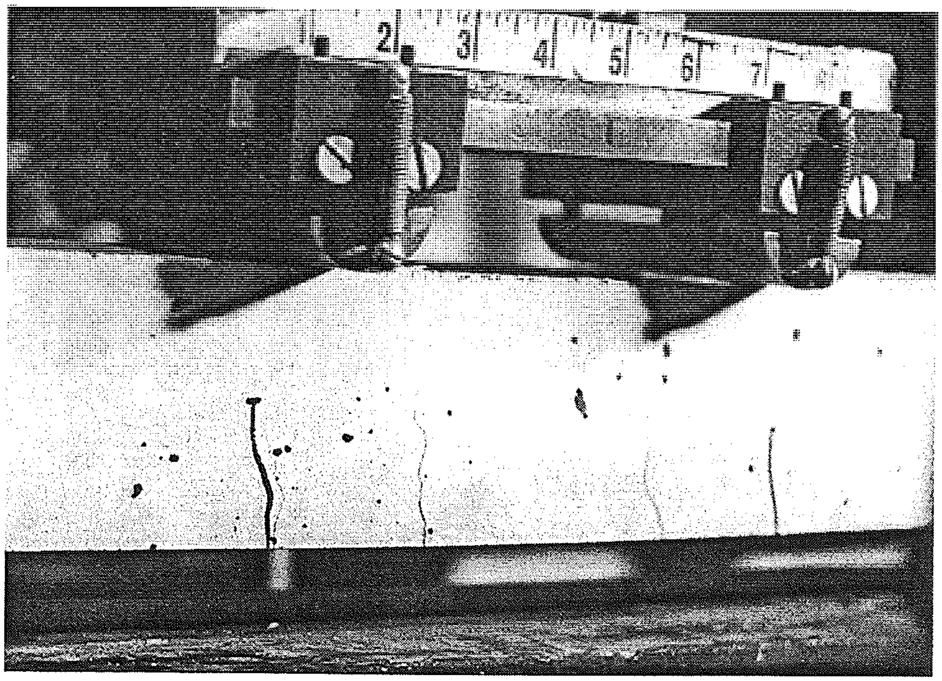

Figure 7. Control Specimens (Flexural Cracks)

3.2 Sika System

Flexure-Delamination: CFRP peeling-off initiated at a flexural crack.

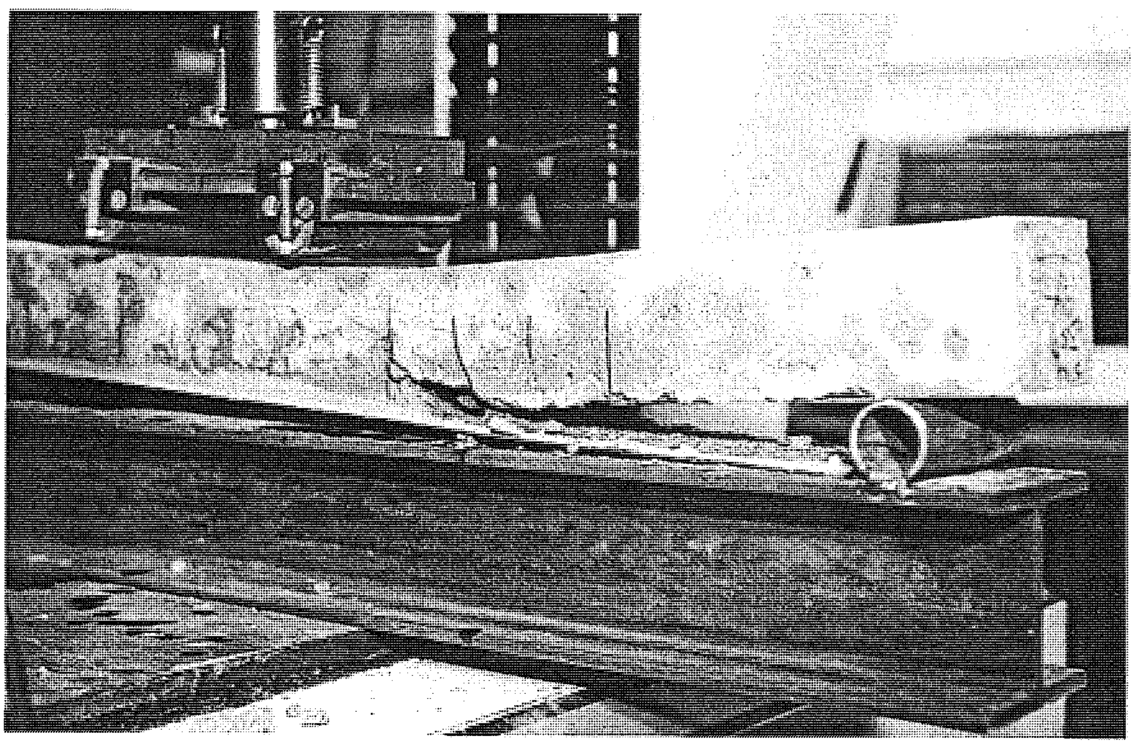

Figure 8. Sika System. Failure Mode: Flexure-Delamination

Shear-Delamination: CFRP peeling-off initiated at a shear crack.

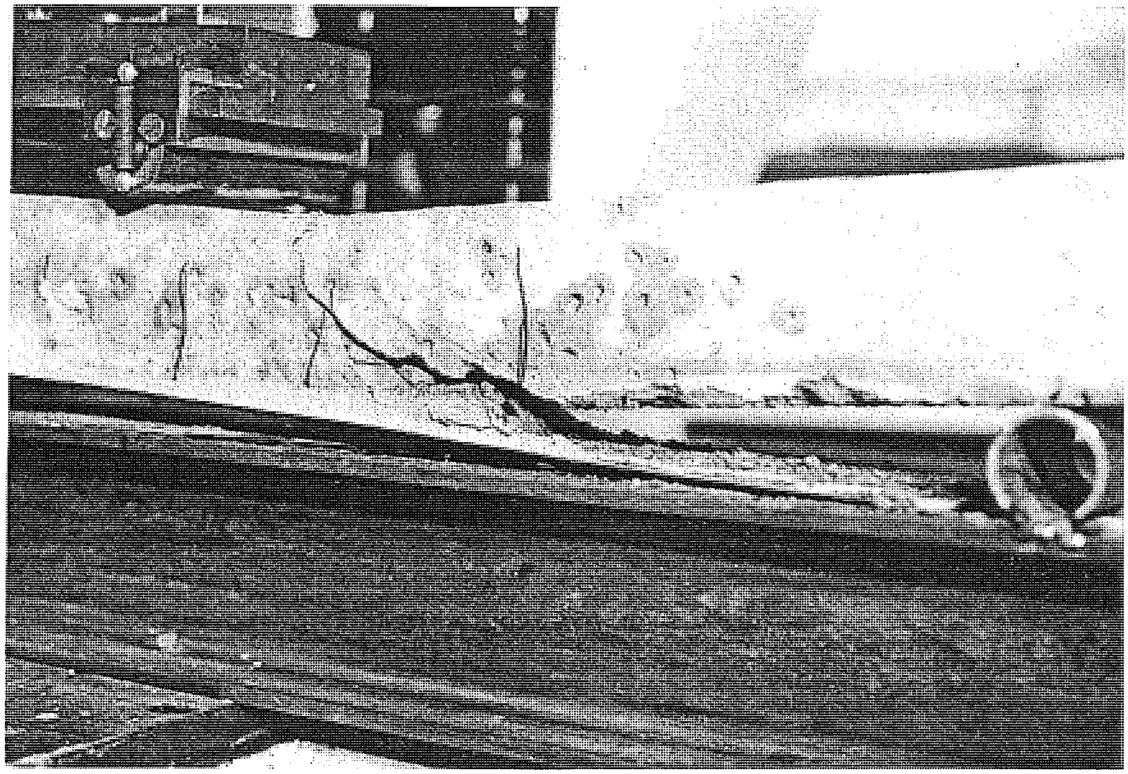

Figure 9. Sika System. Shear-Delamination Failure

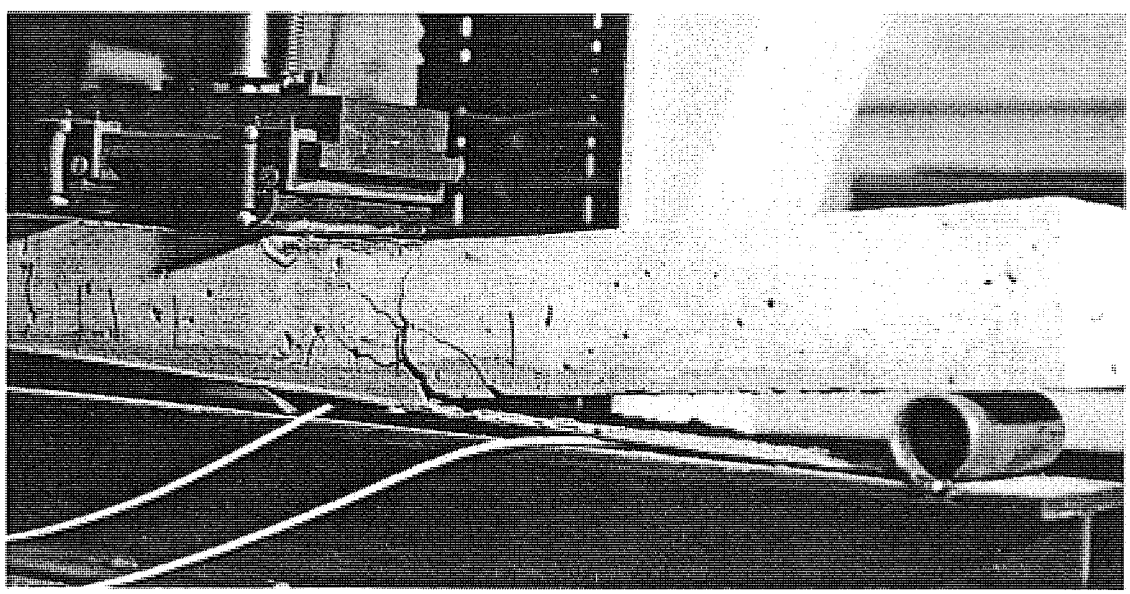

Figure 10. Sika System. Shear-Delamination Failure

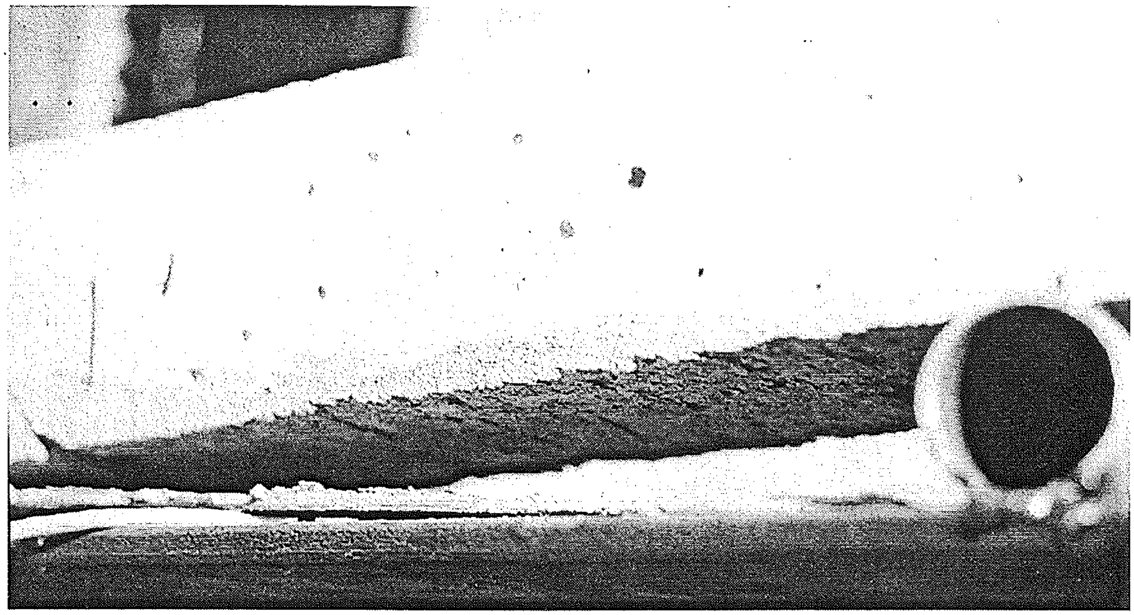

Figure 11. Sika System. Concrete Surface after CFRP Peeling Off

Shear: Vertical shear failure at the end of the CFRP laminate

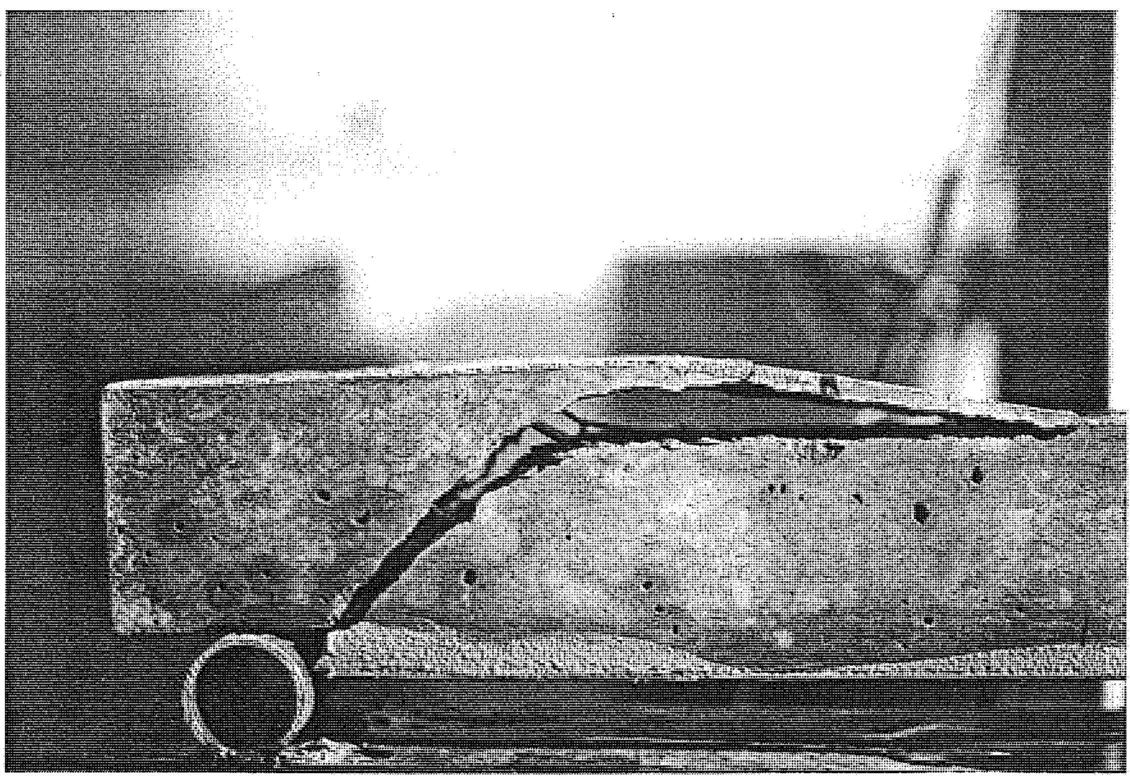

Figure 12. Sika System. Vertical Shear Failure

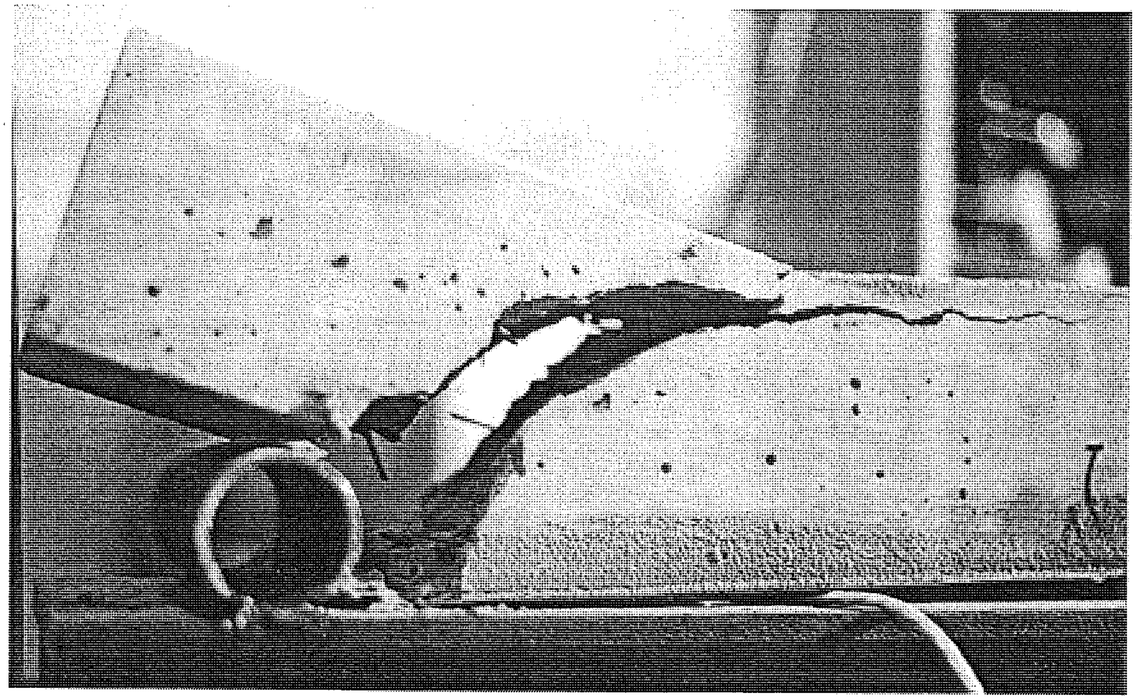

Figure 13. Vertical Failure of a Specimen Strengthened with Sika System

3.3 Tonen System

Flexure-Delamination: CFRP peeling-off initiated at a flexural crack.

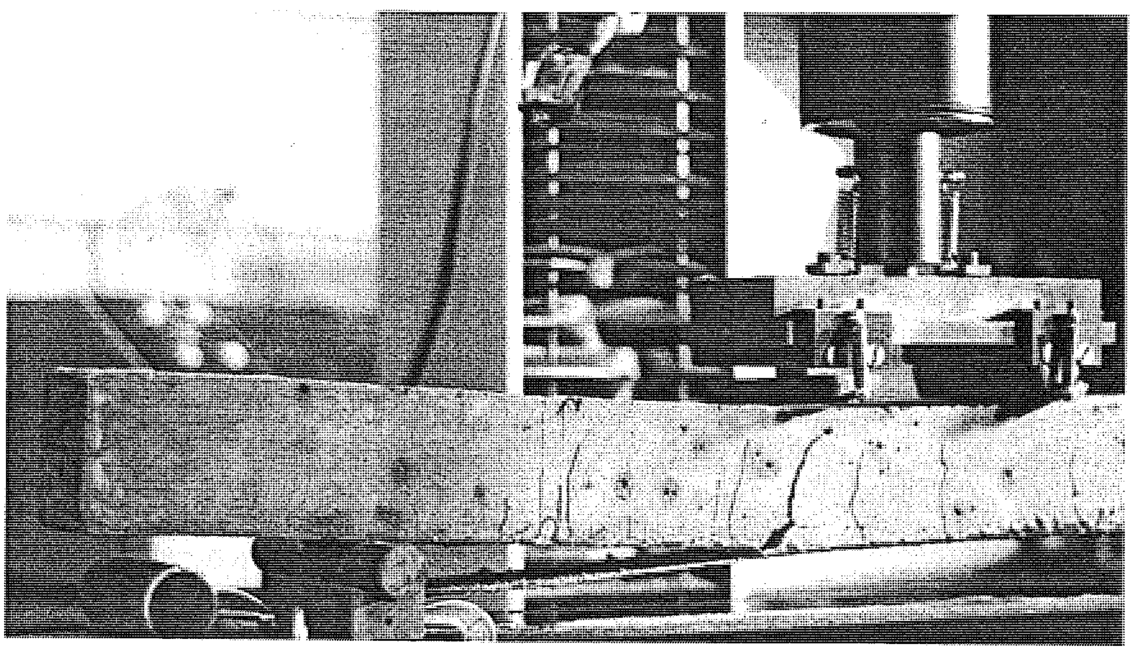

Figure 14. Tonen System. CFRP Peeling-Off Started at a Flexural Crack

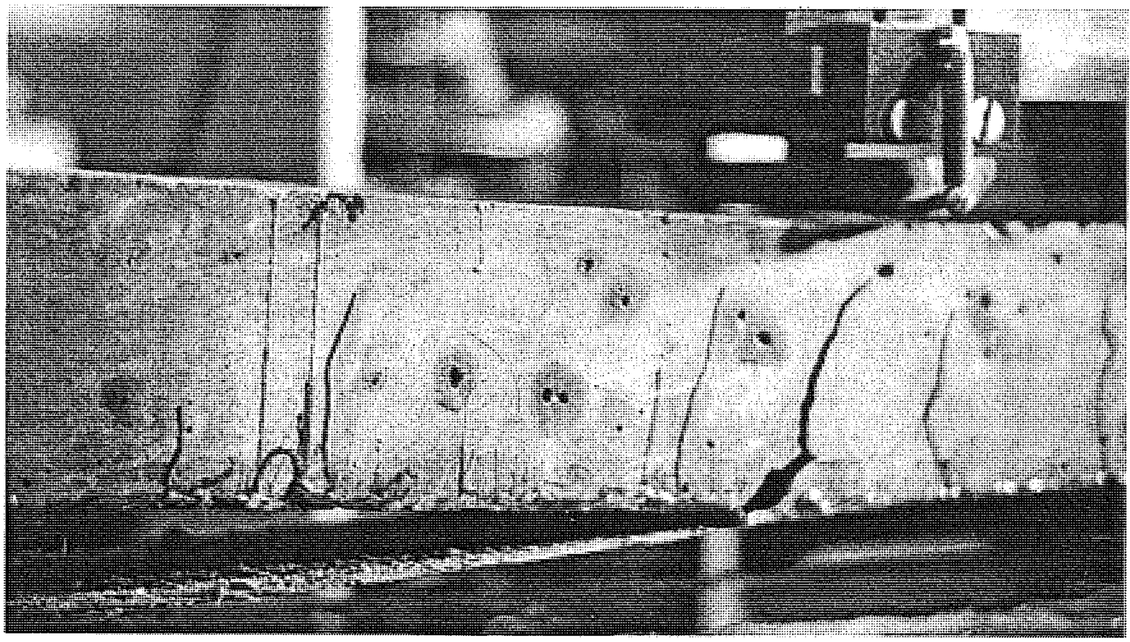

Figure 15. Tonen System. Flexure-Delamination Failure

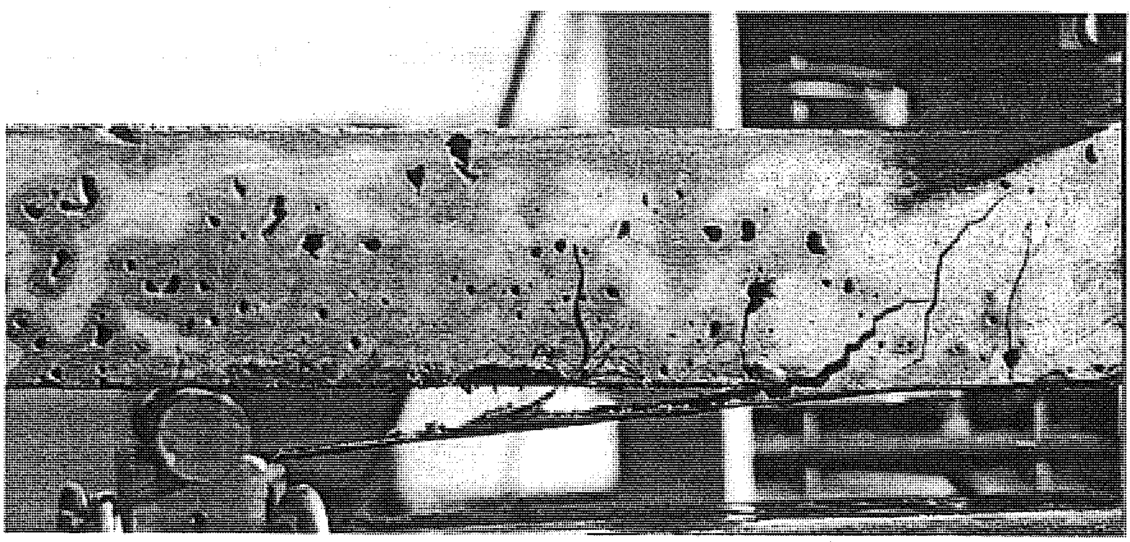

Figure 16. Tonen System. Flexural Cracks and CFRP Peeling-Off

Shear-Delamination: CFRP peeling-off initiated at a shear crack.

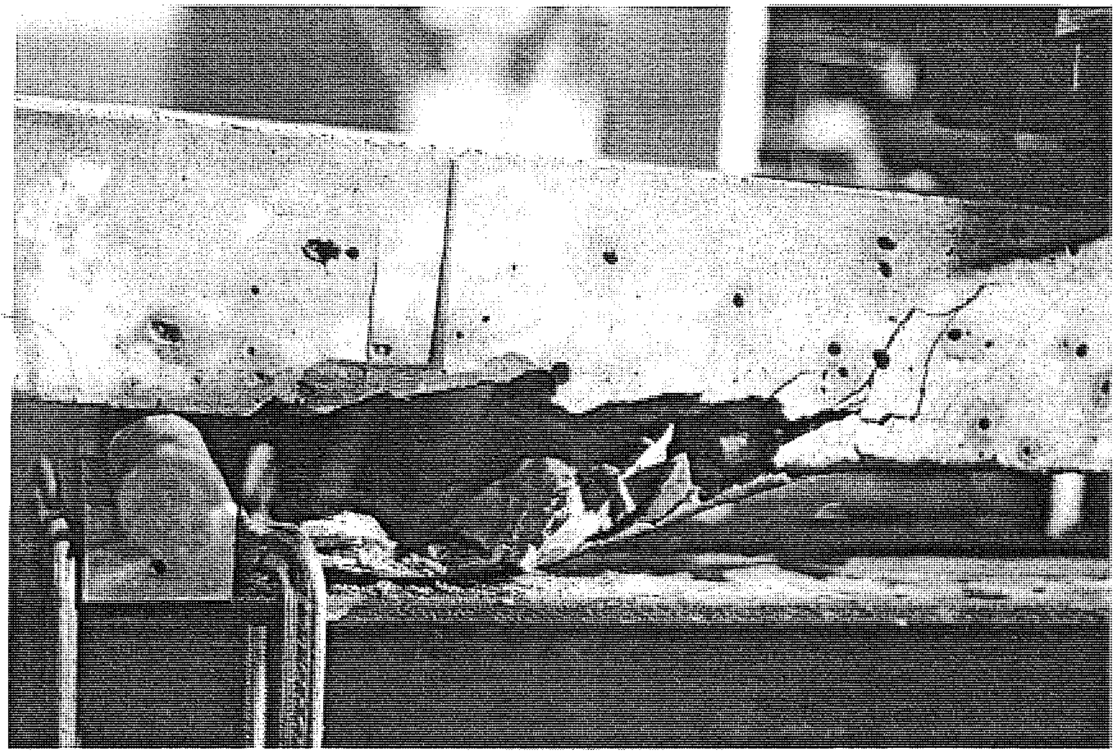

Figure 17. Tonen System. CFRP Peeling-Off Initiated by a Shear Crack

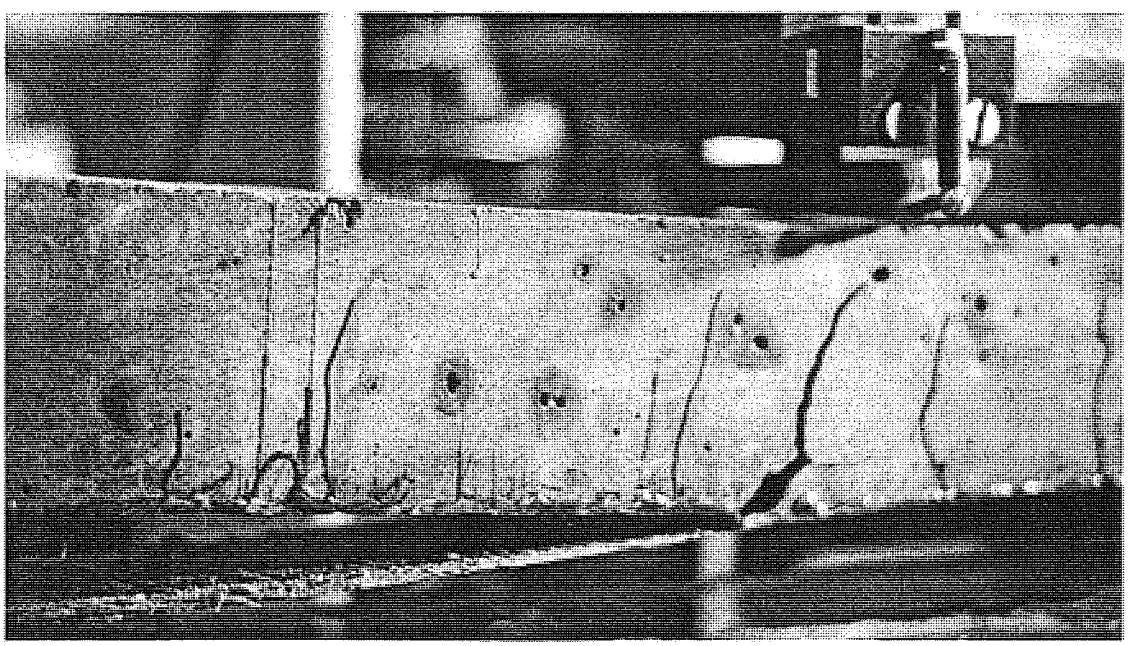

Figure 18. Tonen System. Shear-Delamination Failure

Figure 19. Shear-Delamination Failure of a Specimen with Tonen System

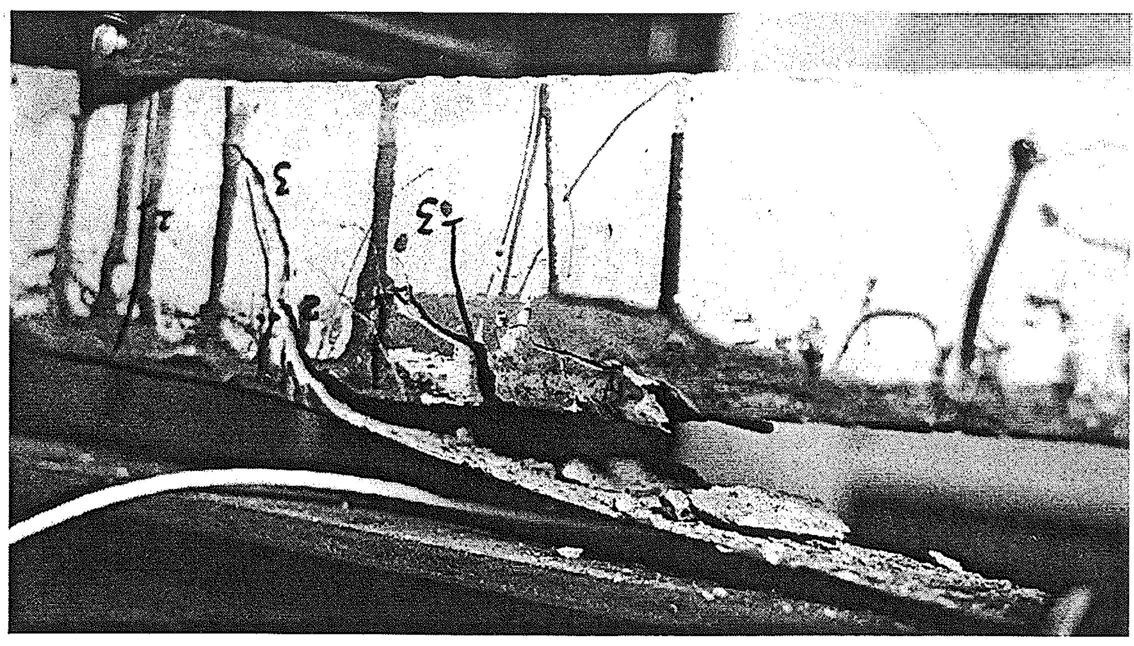

Figure 20. Tonen system. CFRP Peeling-off. Some Pieces of Concrete remained attached to the Laminate

UNIVERSITY OF MICHIGAN

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAMS USING CFRP LAMINATES

Volume 7: Technical Specifications

by

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16. Abstract

Repair and strengthening techniques using glued-on carbon fiber reinforced plastic (CFRP) plates (also called sheets, tow sheets, and thin laminates) form the basis of a new technology being increasingly used for bridges and highway superstructures.

The study described in this report (Volumes 1 to 7) focused on the use of carbon fiber reinforced plastic (CFRP) laminates for repair and strengthening of reinforced concrete beams. Its primary objectives are: 1) to ascertain the applicability of CFRP adhesive bonded laminates for repair and strengthening of reinforced concrete beams; 2) to synthesize existing knowledge and develop procedures for implementation in the field; 3) to identify key parameters for successful design and implementation; and 4) to adapt this technique to the specific conditions encountered in the state of Michigan.

This report consists of 7 volumes:
Volume 1 - Summary Report
Volume 2 - Literature Review
Volume 3 - Behavior of Beams Strengthened for Bending
Volume 4 - Behavior of Beams Strengthened for Shear
Volume 5 - Behavior of Beams Under Cyclic Loading at Low Temperature
Volume 6 – Behavior of Beams Subjected to Freeze-Thaw Cycles
Volume 7 – Technical Specifications

Volume 7 (this volume) provides technical specifications based on information provided by the manufacturers of the two CFRP strengthening systems used and augmented by the experience accrued during the course of this investigation. Since the adhesive-bonded plate repair and strengthening technique applies to plain, reinforced and prestressed concrete structures, as well as steel and timber structures, the experience gained during this project and the technology transfer developed cover a wide range of future applications.

17. Keywords bond, carbon fiber, composites materials, laminates, rehabilitation, reinforced concrete, repair, strengthening systems			18. Distribution Statement No restrictions. This document is available to the public through the Michigan Department of Transportation
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- shear
- strengthening systems

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PREFACE

This project titled: *"Repair and Strengthening of Reinforced Concrete Beams using CFRP Laminates"* is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of the project were:

- To ascertain the applicability of Carbon Fiber Reinforced Plastic (CFRP) glued-on plates for repair and strengthening of concrete beams;
- To synthesize existing knowledge and develop procedures for implementation in the field;
- To adapt this technique to the specific conditions encountered in the state of Michigan.

The project consisted of 8 tasks as follows:

- A report containing a literature review and a comprehensive synthesis of the latest state of knowledge on the glued -on FRP technique (Task 1);
- Laboratory testing and verification of the selected CFRP glued-on technique according to the proposed experimental program: bending (Task 2), shear (Task 3), freeze-thaw (Task 4), temperature and high cyclic amplitude load (Task 5);
- An interim and final report summarizing the experimental results (Task 6). The interim report will cover the bending and freeze-thaw tests;
- A summary of field specifications and "how to" details for implementation in field applications;
- Guidelines for design based on the experience developed from the experimental work (Task 7);
- Field monitoring of application of the technique to one bridge selected by MDOT (Task 8a);
- Bridge testing before and after application of the glued-on plate (Task 8b to be conducted by professor A. Nowak, U of M)

This volume provides technical specifications based on information provided by the manufacturers of the two CFRP strengthening systems used and augmented by the experience accrued during the course of this investigation.

1. INTRODUCTION

The technical specifications presented here are part of a research project at the University of Michigan supported by the Michigan Department of Transportation and the Great Lakes Center for Truck and Transit Research. The project title is "Repair and Strengthening of Reinforced and Prestressed Concrete Beams Using Carbon Fiber Reinforced Plastic (CFRP) Glued-on Plates". The study is aimed at providing experimental verification and recommendations for implementation of a new technology, in which thin fiber reinforced plastic laminates are glued-on the surface of concrete beams in order to strengthen them.

The primary objectives of this project are to ascertain the applicability of CFRP glued-on plates for repair and strengthening of concrete beams, to synthesize existing knowledge and develop procedures for implementation in the field, and to adapt this technique to the specific conditions encountered in the State of Michigan. As part of the tasks of this research project, this report is to present a summary of field specifications and "how to" details for implementation in field applications.

1.1. Limitations

These specifications present the synthesis and the comparison of the application procedures based on the information extracted from different commercial and research sources. In case of use of a particular FRP system, the application procedure provided by the material supplier should be followed.

1.2. Disclaimer

The successful application and use of these technical specifications is sole responsibility of the user and is dependent on the application of sound judgement by qualified professional engineer with a thorough understanding of concrete behavior and structural mechanics.

2. SPECIFICATIONS FOR THE USE OF CARBON FIBER REINFORCED PLASTIC (CFRP) LAMINATES FOR REPAIR AND REHABILITATION OF CONCRETE STRUCTURES

2.1. DESCRIPTION

Thin fiber reinforced plastic laminates, particularly carbon fiber reinforced plastic (CFRP) laminates are externally epoxy bonded to the surface of reinforced and prestressed concrete beams in order to strengthened them. The amount and localization of the CFRP laminate for a particular strengthening solution should be defined by a registered or licensed structural engineer.

2.2. MATERIALS

Materials required for this work are related with the particular strengthening system used. Refer to appendix A, special provision for strengthening with Sika® Carbodur® (taken from Engineering Guidelines for the use of Carbodur ® (CFRP) laminates for structural strengthening) and to appendix B, special provision for strengthening with the MBrace strengthening system (taken from Master Builders Technologies- Restoration products. Specification Bulletin: MBrace™ Composite Strengthening System with carbon fiber reinforcement for concrete substrates).

Properties of currently available commercial composite sheets are summarized in Table 2.1. The characteristics of the proprietary adhesives are summarized in Table 2.2. The information presented was provided by the supplier of the corresponding strengthening system. It should be noticed that MBrace™ CF 130 carbon fiber sheet is the same material as the Forca Tow sheet FTS C-130.

A. Delivery, storage, and handling

Delivery of the specified product(s) must be in original, unopened containers with the manufacturers name, labels, product identification, and batch numbers.

Storage and conditioning of the specified product(s) must be as recommended by the manufacturer.

Table 2.1 Data Summary for Commercial FRP

Property \ Product	Sika *CarboDur*	Tonen *Forca Tow Sheet*		Mitsubishi *Replark*
Type / Grades	Type S512 Type S812 Type S1012	FTS-C1-20 FTS-C 130 (MBrace CF 130) FTS-C 530 (MBrace CF 530)	FTS-GE-30 FTS-GT-30 MBrace EG 30	Type 20 (MRK-M2-20) Type 30 (MRK-M2-30) Type MM (MRK-M4-20) Type HM (MRK-M6-30)
Type of Fibers	Carbon fibers Toray T300 &T700	High tensile CF High tensile CF High modulus CF	E-Glass Fibers T-Glass Fibers E-Glass Fibers	Standard Modulus CF Standard Modulus CF Medium Modulus CF High Modulus CF
Type of Matrix	epoxy resin matrix	epoxy resin matrix	epoxy resin matrix	Epoxy resin matrix
Tensile Strength	2,400 MPa all grades	3,480 MPa 3,480 MPa 2,942 MPa	1,516 MPa 2,694 MPa 1,516 MPa	3,400 MPa 3,400 MPa 2,900 MPa 1,900 MPa
Modulus Of Elasticity ($1000\ N/mm^2 = 1\ GPa$)	155 GPa all grades	230GPa 230GPa 373 GPa	72.6 GPa 87.1 GPa 69.0 GPa	230 GPa 230 GPa 390 GPa 640 GPa
Ultimate Elongation At Break [%]	1.9 all grades	1.5 1.5 0.8	2.1 3.2 2.1	N.A.
Fiber Areal Weight g/m^2	N.A. Fiber volume >68%	200 300 300	300 300 300	200 300 300 300
Density ρ	$1.6\ g/cm^3$	$1.82\ g/cm^3$	$2.55\ g/cm^3$ $2.50\ g/cm^3$	$1.6\ g/cm^3$
Thickness	1.2 mm all grades	0.11 mm 0.17 mm 0.17 mm	0.118 mm 0.120 mm 0.118 mm	0.11 mm 0.17 mm 0.17 mm 0.14 mm
Sheet width Cm	50 mm 80 mm 100 mm	50 cm all grades	50 cm all grades	25 cm all grades 33 cm all grades 50 cm all grades
Proprietary Adhesive	Sikadur 30 - Epoxy resin two components adhesive.	**For FTS C:** FP-NS, FP-NSS, FP-NSW, FP-S, FP-WE7, FP-WE7W, FR-E3P, FR-E3PS, FR-E3PW **For MBrace CF:** MBrace Primer, Putty, and Resin	**For FTS G:** Same as for FTS C1-C5 **For MBrace EG:** MBrace Primer, Putty, and Resin	Epotherm Primer, Putty and Resin
Lengths Available	Any length	Standard: 100 m.	Standard: 100 m.	Standard: 100 m

Note: the values of properties are given in a sequence corresponding to grade sequence.

N.A. = Information not available

Table 2.2 Comparison of Commercial Adhesives

Property \ Producer	Sika	Master Builders		Tonen									Mitsubishi					
Grade	Sikadur 30	MBrace Primer	MBrace Saturant	FP-NS	FP-NSS	FP-NSW	FP-S	FP-WE7	FP-WE7W	FR-E3P	FR-E3PS	FR-E3PW	PS301	PS401	PS318	L525	L700S	L700W
Material	Epoxy	Epoxy	Resin	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy	Epoxy
Type	Adhesiv Mortar	Primer	Curing resin	Primer	Primer Summer	Primer Winter	Primer Penetr.	Primer Summer	Primer Winter	Resin Standard	Resin Summer	Resin Winter	Primer Fall/ Spring	Primer Spring/ Fall	Primer All year	Putty All year	Resin Spring/ Fall	Resin Winter
Appl. Temp. °C	10-38	5-38	10-38	15-25	25-35	5-15	5-35	15-25	5-15	15-25	25-35	5-15	5-25	20-35	5-35	5-35	15-35	5-15
Mixing Ratio: • by weight				2:1	2:1	2:1	1:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1	2:1
• by volume	3:1	3:1	3:1															
Viscosity, cps	N.A.	N.A.	1,600	2,000	2,000	1,300	90	45,000	45,000	20,000	20,000	10,000	500 (MPaS)	500 (MPaS)	17 (MPaS)	Putty	4,000 (MPaS)	3,500 (MPa S)
Pot Life, min		N.A.																
@30°C					40		120				40		25	140	> 120	30	50	
@23°C	70												40	240	> 120	50	70	20
@20°C			30	40	120	20	120	40	20	40	110	20						
@10°C							120		50			50	95		> 120	100	-	70
Setting Time, hrs	N.A.		N.A.															
@30°C											7	7	-	4.0	-	3.0	3.5	-
@23°C													3.5	7.0	2.5	3.5	6.0	3.5
@20°C				10	12	7	3	12	10	10	12	12	-	-	-	-	-	-
@10°C													6.5	-	4.0	5.0	-	6.0
Curing time, days	N.A.		N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.		-
@30°C																	5	-
@23°C																	7	7
@20°C																	-	
@10°C																	-	14
Compress.Strength (14 day), MPa	59.3 (23°C)	N.A.	88	N.A.									N.A.					
Shear Strength, MPa	24.8	17-19 (steel)	N.A.	12.5 MPa									N.A.					
Tensile Strength (7 day), MPa	24.8	13-15.8	78	N.A.									N.A.					
Elong. at Break	1%	20-30%	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Bond Strength: - Concrete to conc. - Concrete to steel	21.3MPa 17.9MPa	17 MPa (steel)	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Shelf Life	2 years		N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Storage Conditions	4-35°C	18-24C	cool-dry	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Color main agent	Light gray	Clear-Amber	Blue	N.A.									Pale yellow		Transp	White	Green and thixotropic liquid	

N.A. = Information not available

2.3. EQUIPMENT

The type of equipment needed according to the strengthening system used, location of the work and structural and environmental conditions will be defined by a specialized contractor. For equipment required to apply Sika® Carbodur® refer to appendix A, for Tonen MBrace strengthening system refer to appendix B.

2.4. EXECUTION

A. Assessment of existing conditions

The determination of strengthening technique should be supported by thorough examination of existing structural and environmental conditions such as:

- Conditions of the member;
- Concrete quality;
- Reinforcement configuration and location;
- Member geometry;
- Load conditions;
- Environmental exposure (road salt, UV light, freeze-thaw).

The inspection of the structure and the designing of repair and/or reinforcement level should be done by civil or structural engineers and/or professionals who are lawfully registered or licensed to do the job.

Complete working drawings shall be submitted by the Contractor for each installation of the beam/slab strengthening. The working drawings shall contain details of the width of the strips used, properties of materials used, joint and end details, and all other information required for the proper installation of the system.

B. Surface preparation

The following represents a synthesis of the surface preparation recommendations taken from commercial information and also from research sources. Before application of a particular system, one should refer to the appropriate commercial specifications. All possible bonding surfaces involved in the strengthening system (concrete, FRP, steel) should be prepared.

<u>Concrete Surfaces</u>

- In order to provide open roughened texture the application of the following surface preparation techniques and/or inspection steps can be necessary:
  - Sandblasting (high pressure sand abrading);
  - High-pressure water washing;

- Bush-hammering (heavy impact method; can only be used if specifically requested by the Engineer in charge of the surface preparation step);
- Disk abrading (grinding).

- For a new concrete structure, the design engineer must specify the minimum compressive strength allowable to apply the corresponding surface preparation.
- Surface must be free of standing water. Some types of adhesives require a dry surface. ASTM D4263 tests moisture in concrete by means of a plastic sheet method [ASTM-2].
- Watch for the formation of condensation (dew point).
- Removal of dust, grease, oils, curing solutions, or mold release agents, impregnations, waxes, foreign particles, disintegrated materials, paints, plasters, wall papers and other bond inhibiting materials (see surface contaminants below).
- Adhesion to exposed aggregates is generally better than to the hardened cement paste. The laitance must therefore be removed and the aggregate must be exposed as gently as possible.
- The surface to be coated must be even (on a length of 2 meters, unevenness may not exceed 10 mm [SCD-1]). Steps and formwork marks must not be greater than 0.5mm.
- Filling of gaps, cavities and uneven portions of structure with appropriate repair mortar specified by the system producer (see surface defects below). According to Sika, an epoxy repair mortar should be applied (e.g., Sikadur 30 with the addition of 1 part sand to make an epoxy mortar) [SCD-1]. At the direction of the Engineer in charge, the adhesive strength of the concrete surface should be verified after preparation by random pull-off testing (ACI 503R, ACI-1). This testing should prove a minimum tensile strength of the adhesive interface specified by the system supplier (according to Sika, minimum tensile strength, 1.4 MPa with concrete substrate failure).
- The optimum roughness of the substrate (0.5-1.0 mm) can be achieved with sandblasting technique [SCD-1]. Protrusions, laitance, remains from formwork, dowels etc. must be removed. Visual check of the treated substrate surface for foreign matter and inclusions in the concrete is part of quality control.

1. **Surface contaminants.** Examples of the more common contaminants are provided [XX-46]:

a. **Curing compounds.** They are liquid solutions that are routinely applied to newly placed concrete surfaces to facilitate hydration curing by retarding water loss from evaporation. Some curing compounds may be compatible with coatings. In case of doubt consult with the supplier of the strengthening system and/or completely remove the curing compounds.

b. **Dust.** Dust may land on a cleaned surface as airborne fallout from industrial pollution or from work being performed in the immediate area. Dust should be removed by vacuum cleaning. An alternative method consists of blowing down clean, oil-free, compressed air. The method of dust removal should be approved by the engineer in charge of the repair work.

c. **Efflorescence.** It is a powdery white and sometimes crystalline deposit of water-soluble salts that migrate to the concrete surface with water. The salts are left on the surface when the water evaporates. Movement of the moisture and dissolved salts will be from the hot to the cold side of a wall. This efflorescence must be removed from the surface to prevent loss of coating adhesion.

d. **Laitance.** This is a weak layer of partially hydrated cement paste at the top surface of concrete. It is usually caused by extended open time during finishing in cooler temperatures and/or overworking (toweling/floating) the finish, and transporting cement fines to the surface. If laitance is not completely removed to sound concrete, it will result in loss of coating adhesion. Laitance may be from 1.6 mm to 3.2 mm thick.

e. **Form release agents.** They facilitate the removal of forms from cured concrete. Form release agents are inherently strong bond breakers. These agents can cause coating adhesion problems if they are transferred to the concrete.

f. **Oil, grease, tar, and gum.** May be found on concrete, usually floors, exposed to foot and vehicular traffic or machinery. These contaminants can penetrate deeply into concrete making effective removal difficult. If not removed they will cause adhesion problems for coatings and patching materials. They can also bleed through the coating to cause staining. Tar, gum, and bitumen-based materials will act the same as oil and grease. Hydraulic fluids and similar materials can penetrate deeply thereby requiring additional cleaning or substrate removal. After applying the corresponding cleaning method, inspection should be performed to establish the acceptability of the substrate for the application of coatings. Oil may be detected by a water break test. Clean potable water should be lightly sprinkled or sprayed (fine mist) on the surface. If the water wets and spreads out instead of beading up, the surface may be considered relatively oil and grease free. Gum may take on the appearance of an oil spot and will cause problems in coating delimitation if not properly removed

2. **Surface defects.**

a. **Fins.** They are high points in the form of ridges or knobs having a relatively sharp surface. Coatings applied over fins will be uneven and low in film thickness due to surface tension pullback of the coating at the fins' edges. Fins and projections should be removed by grinding or stoning to a flat plane surface followed by a light wire brushing to remove dusty laitance, followed in turn by a vacuum or air blast cleaning.

b. **Eggshell.** It is a very thin, sometimes translucent film of laitance and bleed water residues that forms over air pockets and bug holes in the concrete surface. This thin film is very easily broken open exposing the hole beneath. Eggshell areas should be located, broken open, cleaned out, and the resulting hole patched flush with the surrounding surface.

c. **Tie holes.** They are small holes remaining in the concrete surface after the concrete form tie bars have been removed. Tie holes should be cleaned out and patched flush with the surrounding surface.

d. **Sacking or rubbing.** It is the hand application of cement sand mortar extruded through a burlap sack. The mortar sack is rubbed over new concrete. Sacking can cover minor surface defects and fill small voids but it cures into an extremely weak layer having very poor cohesive (tensile) and bond strengths. If present, sacking must be removed prior to the application of any coating system.

3. **Internal contaminants and defects.**

Concrete may also contain chemical contaminants and hidden mechanical defects within itself. These contaminants and resultant defects can cause problems in both coatings and patching material performance. Soluble salts in the form of chlorides, sulfates and nitrates can cause decomposition and cracking of the concrete from strong expansive internal reactions. Chloride-contaminated concrete substantially contributes to reinforcement corrosion as well as concrete that has become carbonated in the immediate area of the reinforcement.

Concrete and masonry deteriorated as a result of the expansive reactions caused by sulfates, nitrates, and other salts must be completely removed. The source of both salts and their water transport mechanisms must be eliminated or significantly reduced to realize future service life.
Repairs due to reinforcement corrosion should always be performed in accordance with the International Concrete Repair Institute (ICRI) guidelines, document No. 037730 "Surface Preparation for the Repair and Deterioration Concrete Resulting from Reinforcing Steel Corrosion" [XX-48]

4. Cleaning and preparation methods

Cleaning and preparation should produce a clean sound surface having a roughness approximately equal to 60 to 80 grit medium sandpaper [XX-46]. Note that some of the methods indicated here may also be specified in the MDOT Standard Specifications for Construction.

a. **Steam cleaning.** Regulated by ASTM D 4258 [ASTM-3] and ASTM D 4261 [ASTM-4]. Steam cleaning machines produce quantities of wet or dry steam directed at the concrete in a high concentration and at a velocity sufficient to loosen, soften and remove the contaminants. Detergents, degreasers, and other chemicals are often added to the water to greatly increase their effectiveness. Their use should be authorized by the Engineer in charge. It should be noted that steam cleaning only removes surface contaminants and not those in the pores of the concrete.
b. **Shot blasting.** Regulated by [4259 [ASTM-5] and ASTM D 4258 [ASTM-3]. Centrifugal shot blasting is a very effective, clean, and dust-free method for removing hardened films of contamination and texturing horizontal concrete without water or chemicals. This process involves impacting the surface with high velocity steel shot abrasive. The shot blasting media is thrown against the concrete from an enclosed high velocity rotating paddle wheel. A separate dust collector then removes the abrasive, dust, and contaminants. The cleaned steel shot is then recycled to the blast wheel where the cycle repeats. Shot blasting provides a relatively uniform texture ranging from fine granular to a coarse sandpaper finish. Steel shot blasting is not very effective for the removal of rubbery elastomeric materials, however, in some instances, it may be used to clean them.
c. **Abrasive blast cleaning.** Blast cleaning is a method for preparing and texturing concrete surfaces by impact with a high velocity stream of fine mineral aggregate abrasives propelled by clean compressed air. Blast cleaning produces a textured, physically sound substrate free of surface contamination and fines. Surface hardness of concrete determines whether this method is applicable for a particular work. Test areas should be tried using the same equipment, air pressure, hose lengths, nozzle size, and abrasive. Sand abrasives should be selected according to the level of cleaning required. Mineral abrasives should have a sharp angular shape and be at least a 6.5 on the Mohs' mineral hardness scale. Abrasives containing free silica can not be used, as they can cause lung disease silicosis. Wet abrasive blast cleaning and equipment may be used when dust abatement is necessary. This abrasive method is not generally effective on rubbery elastomeric materials and should not be used for that purpose.

A comparison of different surface preparation techniques depending on the level of contamination and deterioration and the depth of removal of concrete cover is presented in the Table 2.3 below (based on Sika materials.) The bigger the contamination and the thickness of removed material are the more aggressive is the method that has to be applied.

Steel Surfaces

- Removal of grease, oil, rust and scale;
- Preparation – abrasive blasting;
- Eye inspection for the formation of condensation (dew point);
- If not bonded immediately, the surface has to be protected with compatible corrosion inhibitor.

Table 2.3 Methods of Concrete Surface Preparation

Preferred use for the surface conditions below	Method
Removal of oils, greases, proteins (water-soluble, agents water-emulsive)	Steam jets with added wetting agents
Removal of old paint	Steam jets with added wetting agents and sand
Removal of old paint, heavy contamination on low-strength surface areas of the concrete, damage from the road salts	Abrasive-blasting, Water sand-blasting, High pressure water-jets
Removal of thicker old coatings from deeper areas with low surface strength, deep-reaching contamination	Flame-cleaning and mechanical cleaning
Removal of deep-reaching road salts and other contamination.	Grinding and mechanical cleaning

FRP Sheets

- Place FRP fabric on even surface (such as table).
- Check the material for possible damages, cracks etc.
- For the Sika system, the CFRP strips have to be cut to proper length by metal saw or disk-cutter before application.
- For the Tonen system, the carbon fiber sheet must be cut beforehand into prescribed sizes using scissors and/or cutter.
- Sika recommends to wipe clean with appropriate cleaner (e.g. acetone). This operation removes soiling as well as carbon dust. Cleaning should be continued until white cloth remains white.
- Dry CFRP laminate with a clean rag (Sika).

C. Mixing of Adhesives

- Consult technical data sheet for specific type of adhesive.
- Protect adhesive and components from direct sunlight.
- Maintain appropriate proportions of components (parts by weight or parts by volume).
- Prepare only that quantity which can be used within its pot life.

- Premix each component in the original containers.
- Consult technical data sheet of components 1 and 23 to define which component should be added to. Generally hardening component is added to resin component.
- Mix the adhesive until uniform in consistency (uniform color). Use electric hand mixer for about 3 minutes or mix manually with the trowel or spatula. Mix with low speed so that as little air as possible is entrained (max. 500 rpm).
- Take care to scrape the sides of the pail during mixing.
- Do not use the epoxy resin if it has hardened lumps and high viscosity once the container was opened.
- The pot life of the adhesive begins when the resin and hardener are mixed.
- To obtain longer workability and pot life the following suggestions can be followed:

Pot life is longer at lower temperatures. If necessary the adhesive components can be chilled to room temperature before mixing. Adhesive prepared in smaller quantities has usually longer pot life.

D. Application to Structure

- Do not apply material if it is raining or snowing, or if they appear to be imminent.
- Precautions should be taken to avoid damage to any surface near the work zone due to mixing and handling of the specified material.
- If the system being used requires the primer (Tonen, Mitsubishi), apply according to manufacturer's specifications.
- Apply the mixed adhesive onto the concrete with trowel, float or spatula to form a layer of required thickness.
- The adhesive has to be applied with great care to the concrete surface to ensure that all the voids are filled and no cavities are left.
- Apply the mixed adhesive onto the CFRP laminate to form a layer of required thickness.
- Within the open time of the adhesive, depending on the temperature, place the FRP onto the concrete surface.
- Press the laminate into the adhesive using the hard rubber roller until the adhesive is forced out on the sheet sides (Sika) or forced into the fibers in the sheet (Tonen, Mitsubishi).
- Remove excess adhesive.
- Leave the applied layer undisturbed for at least 24 hours.
- When wrapping a right angled corner, this corner should be ground to be rounded enough to stick thoroughly around the corner.
- Repeat the application process for desired number of layers. Maintain proper width of glue line.
- Apply coating for protective or aesthetic finish (optional).
- During the curing time of the adhesive heavy traffic loads should be regulated in order to avoid a possible diminution of the bond strength of the adhesive to the concrete surface.

E. Additional Limitations and Requirements

- Consider that FRP materials are easily damaged in transport or in the field by cutting, bending, and trampling.

- The minimum age of concrete must fulfill the requirements of a particular system. In most cases, it should be 21-28 days unless special curing and drying conditions are provided.
- Do not thin the adhesive unless specified. Solvents will prevent proper curing.
- CFRP material is a vapor barrier after cure.
- Minimum substrate and ambient temperature depends on type of adhesive used. It has to be checked with data sheet.
- In extreme solar radiation, most cold-setting epoxy adhesives experience a reduction of shear modulus and shear strength.

F. Safety Precautions and First Aid

- Consults the material safety data sheet for the adhesive before application.
- Epoxy resins can cause skin sensitization or irritation (dermatosis) after prolonged or repeated contact. They can also be eye irritant or even cause burns.
- High concentration of vapor may cause respiratory irritation. Therefore adequate ventilation is necessary. Overexposure may affect liver, kidney, and/or central nervous system effects.
- Use of standard precautions such as safety goggles and chemical resistant gloves is recommended. Cover hands with barrier cream before starting work.
- In any case avoid direct contact with epoxy resin and limit exposure to a necessary minimum.
- In case of skin contact with epoxy resin wash immediately and thoroughly with soap and water. Contact physician if symptoms persist.
- In case of eye contact with epoxy resin wash immediately with plenty of water for at least 15 minutes. Immediately contact physician.
- In case of respiratory problems, remove person to fresh air. Contact physician if symptoms persist.
- In case of respiratory overexposure (excess of PELs) use the appropriate, properly fitted NIOSH/MSHA approved respirator.
- When sanding, possible exposure to crystalline silica (sand) dust may cause delayed lung injury and is listed as a suspect carcinogen by NTP and IARC. Use of appropriate dust protection is recommended.
- In case of spills or leaks, wear suitable protective equipment, contain spill, collect with absorbent material and transfer to suitable container. Ensure proper ventilation of the area.
- Uncured material can be removed with approved solvent.
- Cured material can only be removed mechanically.
- Dispose of in accordance with current, applicable local, state and federal regulations.
- Unused adhesives should not be discharged into drains, waterways or ground.
- Keep materials out of reach of children.

G. Commercial Systems - Comparison of Application Procedures

Application procedures for Sika® Carbodur® are introduced on appendix A. Application procedures for Tonen MBrace strengthening system are introduced on appendix B. Table 2.4 provides the comparison of application procedures for the three main strengthening systems evaluated here.

Table 2.4 Comparison of Commercial Application Procedures

Procedure	Sika *SikaDur*	Tonen Forca Tow Sheet	Mitsubishi *Replark*
FRP Cleaning	Yes (required)	Yes (required)	Yes (required)
Concrete Water Jet	Yes (allowed)	Yes (*)	Yes (*)
Concrete Sandblasting	Yes (required)	Yes (*)	Yes (*)
Concrete Grinding	Optional	Yes (required)	Yes (required)
Priming	No	Yes	Yes
Putty (Filler) application	Yes SikaDur 41	Yes	Optional
Undercoating - 1st Resin Coating for 1st and further plies	Yes	Yes	Yes
Protective overcoating - 2nd Resin Coating	No	Yes	Yes
Finishing and Painting	Optional	Optional	Yes

* Application procedure has been performed experimentally.

3. REFERENCES

ACI - American Concrete Institute

ACI-1 ACI 503R. Pull-Out Test of Driven Pins in Concrete.

ASTM – American Standard Specifications

ASTM-2 ASTM D 4263. Indicating Moisture in concrete by the Plastic Sheet Method.

ASTM-3 ASTM D 4258. Surface Cleaning Concrete for Coating.

ASTM-4 ASTM D 4261. Surface Cleaning Concrete Unit Masonry for Coating.

ASTM-5 ASTM D 4259. Abrading Concrete.

SCD – Sika (Carbodur)

SCD-1 Steiner W., "Strengthening of Structures with CFRP Strips", Advanced Composite Materials in bridges and Structures, El-Badry Editor, M., The Canadian Society for Civil Engineers, Montreal, Canada 1996, pp. 407-419.

Other Research Institutions

XX-46 Blaschko M., Niedermeier R., and Zilch K., "Bond Failure Modes of Flexural Members Strengthened with FRP", Proceeding of the Second International Conference on Composites in Infrastructure, H. Saadatmanesh and M.R. Ehsani, Editors, Tucson, AZ 1998, pp. 315-327.

XX-48 International Concrete Repair Institute (ICRI) Guidelines No. 03730. Surface Preparation Guidelines for the Repair of Deteriorated Concrete Resulting from Reinforced Steel Corrosion.

APPENDIX A: SPECIAL PROVISION FOR USING SIKA® CARBODUR® (CFRP) LAMINATES FOR STRUCTURAL STRENGTHENING

GENERAL

A. Description

This work shall consist of furnishing and installing Carbon Fiber Reinforced Plastic (CFRP) sheets to repair concrete bridge beams as shown on the plans.

B. Work including

1. Existing concrete shall be repaired and reinforced with dry, unidirectional carbon fiber fabric sheet.

2. The work is deemed to include furnishings of materials, labor, and equipment and all items necessary for repair and reinforcing of the concrete as specified on the contract drawings and specifications, complete.

3. Inspect the structural members to be reinforced with Carbon Fiber Reinforced Plastic (CFRP) on the contract drawings to check the location of and inspect cracks, and actual conditions of beams.

4. Install CFRP laminates to reinforce concrete members.

C. Submittals

1. Contractor's qualifications.

2. The epoxy/composite supplier shall submit product data indicating product standards, physical and chemical characteristics, technical specifications, limitations, installation instructions, maintenance instructions and general recommendations regarding each material.

3. The epoxy/composite supplier shall provide testing information to demonstrate system properties of material to be used.

4. The epoxy/composite supplier shall provide a two-year proven record of performance of beam/slab strengthening with carbon fiber materials, confirmed by actual field tests and five successful installations.

5. The epoxy/composite supplier shall provide field supervision specifically trained in the installation of CFRP laminates.

6. Samples of all materials to be used, each properly labeled as specified in MATERIALS.

7. Manufacturer's MSDS for all materials to be used.

8. Certifications (in time to prevent delay in the work) by the Producers of the materials that all materials supplied comply with all the requirements and standards of the appropriate ASTM and other agencies.

D. Project Record Documents

Working Drawings: Complete working drawings shall be submitted by the Contractor for each installation of the beam/slab strengthening. The working drawings shall contain details of the width of the strips used, properties of the materials used, joint and end details, and all other information required for the proper installation of the system.

E. Quality Control

1. Manufacturer/Contractor Qualifications

a. Materials Manufacturer/Supplier: Company specializing in the manufacturing of the products specified in this section with documented experience.

b. Materials Manufacturer/Supplier: Company must be certified by independent audit as ISO 9001.

c. Materials Manufacturer/Supplier: Company shall have in existence for a minimum of 10 years, a program of training, certifying, and technically supporting a nationally organized Approved Contractor program with annual re-certification of its participants.

d. Contractor qualifications: Contractor shall be an approved Contractor of the manufacturer/supplier of the specified product, who has completed a program of instruction in the use of the specified material, and provide a notarized certification from the manufacturer attesting to their Approved Contractor status.

e. A manufacturer's representative is required on site for initial placement of the strengthening system. Direction of the representative must be followed.

2. Quality Control

The contractor shall conduct a quality control program that includes, but is not limited to the following:

a. Inspection of all materials to assure conformity with contract requirements and that all materials are new and undamaged.

b. Inspection of all surface preparation prior to CFRP laminate application.

c. Inspection of work in progress to assure work is being done in accordance with established procedures and established Manufacturer's instructions and specified Engineer instructions.

d. Inspection of all work completed including sounding all repairs to check for debonding and correction of all defective work (see section EXECUTION, part J: quality control and Inspection).

F. Product Delivery, Storage and Handling

Deliver materials clearly marked with legible and intact labels with Manufacturer's name and brand name, product identification and batch number.

The product shall be in original, unopened containers.

Store materials in areas where temperatures conform with Manufacturer's recommendations and instructions.

G. Job Conditions

1. Environmental Conditions. Do not apply material if it is raining or snowing, or if they appear to be imminent. Follow manufacturer's recommendations concerning specific environmental conditions such as concrete and air temperature, dew point and humidity values (see see section EXECUTION, part A, E and F).

2. Protection: Precautions should be taken to avoid damage to any surface near the work zone due to mixing and handling of the specified material.

3. Work only in areas permitted by the Owner approved schedule.

4. Remove all tools, buckets and materials from work areas and store neatly at an approved location daily at the end of work.

5. Protect adjacent areas from damage and stains with appropriate barriers and masking. Repair all damage as a result of the work to its condition at the start of work, or if such cannot be determined, to its original condition.

6. Compliance with OSHA and all other safety laws and regulations is the exclusive responsibility of the Contractor.

H. Technical Support

The Contractor shall provide the services of a trained field representative at the work site at all times to instruct the work crew in the CFRP applications procedures.

1. The manufacturer's Field Representatives must be fully qualified to perform the work.

2. The contractor shall be completely responsible for the expense of the services of the required Manufacturer's Field Representative and the contract price shall include full compensation for all costs in connection therewith.

MATERIALS

A. Carbon fiber reinforced plastic (CFRP) laminates

Sika Carbodur system, as supplied by Sika corporation, Lyndhurst, NJ is considered to conform to the requirements of this specification.

B. Epoxy resin adhesive

Sikadur 30, as manufactured by Sika Corporation, Lyndhurst, NJ, is considered to confirm to the requirements of this specification.

C. Substitutions

The use of other than the specified products will be considered, providing the contractor requests their use in writing to the Engineer. This request shall be accompanied by (a) A certificate of compliance from an approved independent testing laboratory that the proposed substitute products meet or exceed the specified performance criteria (see Table 1.A below), tested in accordance with the specified test standards (see Table 1A below); and (b) Documented proof that the proposed substitute products have a two year proven record of performance of beam/slab strengthening with carbon fiber materials, confirmed by actual field tests and five successful installations that the Engineer can investigate.

EXECUTION

A. Preparing the substrate

The concrete surface must be clean and sound. It may be damp or dry, but free of standing water and frost. Remove dust, laitance, grease, curing compounds, impregnations, waxes, foreign particles, disintegrated materials and other bond inhibiting materials from the surface. The substrate shall first be thoroughly inspected and any unsound concrete must be removed. Prepare substrate by abrasive blasting or other mechanical means. All damaged areas (e.g. cracks, bug holes or surface defects) shall be repaired prior to placing Sika CarboDur. Cracks shall be repaired using a structural injection resin and surface defects shall be filled and leveled with an appropriate repair mortar (e.g. Sikadur 30 with the addition of 1 part sand or Sikadur 43 Patch-Pak). Leveled surfaces shall not deviate more than 3 mm every 300 mm. The surface profile shall not have deviations larger than 3 mm. The adhesive strength of the concrete must be verified after surface preparation by random pull-off testing [ACI-1] at the discretion of the Engineer. Minimum pull off tensile strength requirement is 1.38 MPa.

Table 1A. Performance Criteria

	Property	Requirement
Carbon Fiber Reinforced Plastic (CFRP)	Ultimate tensile strength in longitudinal direction of fiber (ASTM D-3039)	2400 MPa
	Modulus of elasticity (ASTM D-3039)	150,000 MPa
	Elongation at break (ASTM D-3039)	>1.4%
	Fiber volumetric content	>60%
	Apparent density	1.6 g/cm²
	Temperature resistance	500°C
	Thickness	1.2 mm
	Shelf life	Unlimited (with no exposure to direct sunlight)
Epoxy resin adhesive	Compressive strength (ASTM D-695)	62 MPa
	Tensile strength (ASTM D-638)	30 MPa
	Shear strength (ASTM D-732)	17 MPa
	Adhesive strength on concrete (ASTM C-882)	2 MPa
	Modulus of elasticity (ASTM D-638)	12,800 MPa
	Pot life (at 24°C)	70 minutes, minimum
	Density (A+B)	1.77 Kg/l

B. Unpacking Sika Carbodur laminate

CarboDur will normally arrive on site packed as a coil in a box. The contractor has the option of ordering material as custom cut lengths or as one continuous length to be cut at the job site. Once the lid is removed from the box, the coil of CarboDur must be handled carefully to facilitate controlled uncoiling. Care must also be used to avoid splitting the ends of the strip. Since loose carbon fibers may be present on the surface of Sika CarboDur, gloves, mask and goggles are recommended when handling the material.

C. Cutting the CarboDur laminate.

Carbodur laminates should be cut with tools using a "shearing" force (e.g. guillotine or heavy duty shears). Care must be taken to support both sides of the laminate when cutting. As an alternate method, a hacksaw or other abrasive cutting methods may be used. However, extra care must be taken to support the Carbodur laminate on both sides to avoid splintering. In addition, extra care must also be taken to avoid exposure to airborne carbon dust generated while cutting (i.e. use of NIOSH/MSHA mask, and gloves is recommended).

D. Preparation of laminate

Surface shall be wiped clean using appropriate cleaner. Using a clean white cloth wipe down the side which is to receive adhesive (this side is not labeled) with acetone until all residual carbon dust is removed (i.e. the white cloth remains white after wiping the laminate). In case where the design requires 'stacking' of the strips the bottom surface of the strip (which is labeled) shall be lightly sanded prior to the application of the second strip.

E. Mixing Sikadur 30

The ambient temperature and temperature of the epoxy components shall be between 10 and 38 degree Celsius at the time of mixing.

Premix A&B components. Proportion 1 part of component 'B' to 3 parts of component 'A' in a clean pail and mix thoroughly for 3 minutes using a Sika paddle on low speed (400-600 rpm) until all the colored streaks have disappeared. Take care to scrape the sides of the pail during mixing. Mix only that quantity which can be used within its pot life.

Components that have exceeded their shelf life shall not be used.

F. Application of Sikadur 30 to the substrate

The substrate temperature must be above 5°C at the time of application of the Sikadur 30. There should be no standing moisture (glistening) when the epoxy is applied. The moisture content of the surface must be less than 5%.

Apply neat Sikadur 30 to the substrate as a first coat using a spatula to form a uniform thickness of 1.6 mm and a width approximately 13 mm wider than the strip that is to be used.

G. Application of Sikadur 30 to the laminate

Clean the Carbodur strip (roughened side) with an appropriate cleaner (e.g. acetone). Dry the CFRP laminate with a clean rag. Apply neat Sikadur 30 to the side of the CarboDur strip opposite of the side labeled "KLEBERFEI" using a roof-shaped spatula to a nominal thickness of 1.6 mm. The best method to accomplish this is to fabricate a 'hopper' for the Sikadur 30 with the spatula at one end. The CarboDur strip is then pulled through the hopper under the Sikadur 30 and then past the roof-shaped spatula to produce a uniform cross section.

H. Applying CarboDur to the prepared substrate

After the CarboDur laminate and the substrate have been prepared with Sikadur 30, the strip is placed on the concrete (epoxy to epoxy). A rubber roller is then used to properly seat the strip using enough pressure so that the Sikadur 30 gel is force out on both sides of the laminate and so that the glue line does not exceed 3 mm. Excess gel should then be carefully removed. Do not disturb material for 24 hours following application Sikadur 30 will reach its designed strength in 7 days. High-build pigmented epoxy (Sikagard 670W) shall be applied as an overcoat for protection of the system against aggressive environments. A high-performance façade coating

(Sikagard 670W) shall be used for aesthetics and UV protection. Overcoat and topcoat color shall be gray.

I. Cleaning

- Uncured epoxy resins can be cleaned from tools with an approved solvent.
- Cured epoxy resins can only be removed by mechanical means.

J. Quality Control and Inspection

1. A qualified manufacturer's representative shall observe all aspects of onsite material preparation and application, including surface preparation, resin component mixing, application of Sikadur resin and Carbodur plate, curing of composite, and the application of protective coatings.

2. Inspection for Voids/Delaminations. After allowing at least 24 hours for initial resin cure to occur, perform a visual and acoustic tap test inspection of the layered surface. Voids larger than 160 mm^2 shall be repaired. If the total delamination area exceeds 10% of the bond surface area, all the voids shall be marked for repair. Treatment of the delaminated areas shall be done under direction of the qualified manufacturer's representative.

3. Report. The representative shall submit report to the Engineer. Contents shall include both in-process and final inspection details. Contents (as suggested in a private communication with the manufacturer's representative) shall include but may not be limited to :

- Ambient conditions at application.
- Tap test-scanning nap.
- Number and directions of plies applied.
- Cured resin cups.
- Laminated/cured FRP panels.
- Results of tension and pull off tests
- Surface hardness readings.
- Laminate thickness.

MEASUREMENTS AND PAYMENT METHODS

Payment for the CFRP sheets on concrete beams will be based on the area of concrete surfaces covered by the sheet and shall include all costs to furnish and install all layers of the sheet, including materials and labor. The cost of preparing the concrete surface for application of the FRP wrap is included in the payment. Individual layers of the sheet will not be paid for separately.

Item	Pay Unit
Carbon FRP Sheet	Square meter

Chipping and patching of deteriorated beams shall be paid for separately.

APPENDIX B: SPECIAL PROVISION FOR MBRACE COMPOSITE STRENGTHENING SYSTEM

GENERAL

A. Description

This work shall consist of furnishing and installing Carbon Fiber Reinforced Plastic (CFRP) sheets to repair concrete bridge beams as shown on the plans.

B. Work including

1. Existing concrete shall be repaired and reinforced with dry, unidirectional carbon fiber fabric sheet.

2. The work is deemed to include furnishings of materials, labor, and equipment and all items necessary for repair and reinforcing of the concrete as specified on the contract drawings and specifications, complete.

3. Inspect the structural members to be reinforced with Carbon Fiber Reinforced Plastic (CFRP) on the contract drawings to check the location of and inspect cracks, and actual conditions of beams.

4. Install CFRP laminates to reinforce concrete members.

C. Submittals

1. Contractor's qualifications.

2. The epoxy/composite supplier shall submit product data indicating product standards, physical and chemical characteristics, technical specifications, limitations, installation instructions, maintenance instructions and general recommendations regarding each material.

3. The epoxy/composite supplier shall provide testing information to demonstrate system properties of material to be used.

4. The epoxy/composite supplier shall provide a two-year proven record of performance of beam/slab strengthening with carbon fiber materials, confirmed by actual field tests and five successful installations.

5. The epoxy/composite supplier shall provide field supervision specifically trained in the installation of CFRP laminates.

6. Samples of all materials to be used, each properly labeled as specified in MATERIALS.

7. Manufacturer's MSDS for all materials to be used.

8. Certifications (in time to prevent delay in the work) by the Producers of the materials that all materials supplied comply with all the requirements and standards of the appropriate ASTM and other agencies.

D. Project Record Documents

Working Drawings: Complete working drawings shall be submitted by the Contractor for each installation of the beam/slab strengthening. The working drawings shall contain details of the width of the strips used, properties of the materials used, joint and end details, and all other information required for the proper installation of the system

E. Quality Control

1. Manufacturer/Contractor Qualifications

a. Materials Manufacturer/supplier Company must be specialized in the manufacturing of the products specified in this section.

b. Materials Manufacturer/Supplier Company must have been in business for a minimum of 5 years, with a program of training and technically supporting a nationally organized Contractor Training Program.

c. Contractor shall be a trained Contractor of the manufacturer/supplier of the specified product, who has completed a program of instruction in the use of the specified material, and provide a notarized certification from the manufacturer attesting to their Approved Contractor status.

d. A manufacturer's representative is required on site for initial placement of the strengthening system. Direction of the representative must be followed.

2. Quality Control

The contractor shall conduct a quality control program that includes, but is not limited to the following:

a. Inspection of all materials to assure conformity with contract requirements and that all materials are new and undamaged.

b. Inspection of all surface preparation prior to CFRP laminate application.

c. Inspection of work in progress to assure work is being done in accordance with established procedures and established Manufacturer's instructions and specified Engineer instructions.

d. Inspection of all work completed including sounding all repairs to check for debonding and correction of all defective work (see section EXECUTION, pat J.: repair of defective work).

F. Product Delivery, Storage and Handling

Deliver materials clearly marked with legible and intact labels with Manufacturer's name and brand name, product identification and batch number.

The product shall be in original, unopened containers.

Store materials in areas where temperatures conform with Manufacturer's recommendations and instructions.

G. Job Conditions

1. Environmental Conditions. Do not apply material if it is raining or snowing, or if they appear to be imminent. Follow manufacturer's recommendations concerning specific environmental conditions such as concrete and air temperature, dew point and humidity values.

2. Protection: Precautions should be taken to avoid damage to any surface near the work zone due to mixing and handling of the specified material.

3. Work only in areas permitted by the Owner approved schedule.

4. Remove all tools, buckets and materials from work areas and store neatly at an approved location daily at the end of work.

5. Protect adjacent areas from damage and stains with appropriate barriers and masking. Repair all damage as a result of the work to its condition at the start of work, or if such cannot be determined, to its original condition.

6. Compliance with OSHA and all other safety laws and regulations is the exclusive responsibility of the Contractor.

H. Technical Support

The Contractor shall provide the services of a trained field representative at the work site at all times to instruct the work crew in the CFRP applications procedures.

1. The manufacturer's Field Representatives must be fully qualified to perform the work.

2. The contractor shall be completely responsible for the expense of the services of the required Manufacturer's Field Representative and the contract price shall include full compensation for all costs in connection therewith.

I. Acceptable Manufacturers/Suppliers

The following vendors shall be used:

1. CFRP laminates: (Dry, unidirectional sheet only). MBrace Fiber Reinforcement Systems supplied by Master Builders, Inc. 23700 Chagrin Blvd., Cleveland, OH 44122 216-831-5500, 800-MBT-9990, Fax: 216-831-6910.

Epoxy resin adhesive: an approved epoxy system for application of MBrace Composite System.
The epoxy system shall include:
Primer
Base Coat/Filler
Saturant
Topcoat
Submit proposed products to Engineer for approval, including thickness to be applied.

2. Substitutions: No substitutions allowed, except as requested by the Manufacturer/Supplier of the product and the Engineer.

MATERIALS

A. Carbon fiber reinforced plastic (CFRP) laminates

MBrace Fiber Reinforcement Systems supplied by Master Builders, Inc. are considered to conform to the requirements of this specification.

B. MBrace Primer

The MBrace clear epoxy primer is a low viscosity, 100%solids epoxy compound. When applied to concrete the surface is upgraded to give high tensile bond strength to the system being used. Use MBrace primer in compounds for filling air voids and bug holes in concrete walls.

C. MBrace Putty

MBrace putty adhesive is a 100%solids, nonsag paste epoxy material. Recommended uses include sealing surfaces prior to epoxy injection, bonding of rigid materials, and leveling uneven surfaces prior to application of the MBrace system.

D. MBrace Fiber Reinforcement Systems and MBrace Saturant resin

MBrace Fiber reinforcement materials are enveloped in MBrace saturant resin to yield a range of high performance features.

E. MBrace Topcoat

MBrace Topcoat is a decorative and protective coating for metal and concrete in environments where moderate to severe corrosion conditions exist.

EXECUTION

A. General preparation for Application

1. Ambient temperature. Conditions of CFRP process application must be examined carefully during the winter season and/or cold zones. Do not apply CFRP when ambient temperatures are lower than 5 degrees C. Auxiliary heat may be applied to raise surface and air temperature to a suitable range. Utilize "clean" heat source (electric, propane) so as not to contaminate bonding surfaces. Follow manufacturer's recommendations concerning specific environmental conditions such as concrete and air temperature, dew point and humidity values.

2. Condensation. Presence of moisture may inhibit adhesion of primer and/or resin. Do not apply CFRP when rainfall or condensation is anticipated.

3. Handling of Primer and Resin. Refer to Manufacturer's specifications. Do not dilute primer and resin with organic solvent. After the resin has been mixed with hardener, the mixed resin batch must be used within its batch-life. The mixed batch resin must not be used after expiration of its batch-life as increased resin viscosity will prevent proper impregnation of CFRP sheet.

4. Handling of Carbon Fiber (CFRP) Sheet. CFRP Sheet must not be handled roughly. CFRP sheet must stored either by being rolled to a radius greater than 300 mm or by being stacked after cutting. When multiple lengths of CFRP Sheet are adhered to a concrete surface, a 100 mm overlapping length must be applied in the longitudinal (fiber) direction. No overlapping is required in the lateral direction. For multiple overlaps, the lapping areas shall be distributed through the surface and shall not be coincident through the thickness.

B. Preparing the substrate

Disc sander, abrasive blasting or scarification to remove laitance and surface contaminants are required. Concrete must be thoroughly cured, free of oils, curing solutions or mold release agents, dust and must be dry at time of application. The moisture content of the surface, determined by ASTM D 4263, must be less than 5%.

Any steel reinforcement shall be cleaned and prepared thoroughly by abrasive cleaning, and the area patched prior to installation of CFRP laminates. Any deteriorated concrete or corroded reinforcing steel must be repaired according with the International Concrete Repair Institute (ICRI) guidelines, document No. 037730 "Surface Preparation for the Repair and Deterioration Concrete Resulting from Reinforcing Steel Corrosion". Do not cover corroded reinforcing steel with CFRP.

Leveled surfaces shall not deviate more than 3 mm every 300 mm. The surface profile shall not have deviations larger than 3 mm. Existing uneven surfaces must be filled with an appropriate cement based repair mortar. Protuberances greater than 1mm shall be ground or chiseled off. The strength of the interface between the repair mortar and the original concrete shall be at least 1.4 MPa using ASTM D-4541. The compression strength of the repair mortar shall be at least the same of the original concrete. MBrace Putty shall be used to fill small depressions and bug holes.

Prior to initiating surface preparation procedures, the Contractor shall first prepare a representative sample area. The sample area shall be prepared in accordance with the requirements of this specification, and shall be used as a reference standard depicting a satisfactorily prepared surface.

C. Application of MBrace Putty

Mechanically premix MBrace Putty resin (part A) individually and prior to adding hardener. After initial mixing, add MBrace Putty hardener and mix three minutes or until homogeneous. Apply the MBrace Putty adhesive to the substrate using a spring-steel trowel or other suitable implement. The application thickness and subsequent coverage rates will be highly dependent on the condition and profile of the concrete substrate. Surfaces shall be topcoated within two days to assure proper adhesion of the MBrace system to the substrate.

D. Application of Primer

Mix 3 volumes MBrace Primer Part A to 1 volume MBrace primer Part B. Mix for about 3 minutes. Agitation shall be by means of electric hand mixer. Use brush or short nap roller to apply two coats of MBrace Primer. Coverage may vary depending on the density of concrete. Alternatively, the primer may be spray applied with airless spray equipment, followed immediately by thorough back rolling to work the primer into the concrete surface. The primer shall be applied uniformly in sufficient quantity to fully penetrate the concrete and produce a nonporous film in the surface not to exceed 50 micrometers in thickness after full penetration.

MBrace primer cures slowly at lower temperatures. Refer to manufacturer for working time, recoat time and time to be able to sustain traffic. Surface irregularities caused by primer coating must be ground and removed using disk sander, etc. if any minor protrusions on the concrete surface still remain, such surface defects may be corrected again using epoxy resin base coat-filler as needed.

When the primer coat has been left unattended for more than one week after the application, the surface of the primer coat shall be roughened using sandpaper. Do not wipe with solvent.

E. Cutting the laminate.

Carbon fiber sheet must be cut beforehand into prescribed sizes using scissors and/or cutter. The size of CFRP sheet to be cut is preferably less than 3 m in length, but may be longer if access allows.

F. Preparation of laminate

Surface should be wiped clean using appropriate cleaner. Using a clean white cloth wipe down the side which is to receive adhesive (this side is not labeled) with acetone until all residual carbon dust is removed (i.e. the white cloth remains white after wiping the laminate). In case where the design requires 'stacking' of the strips the bottom surface of the strip (which is labeled) should be lightly sanded prior to the application of the second strip.

G. Mixing MBrace Saturant resin

Mechanically premix the MBrace saturant resin (part A) individually prior to adding hardener. After initial mixing, add MBrace saturant hardener (part B) and mix one minute or until homogeneous. The mix ratio is 3 to 1 (parts A to part B) by volume.

H. Application of MBrace Saturant resin to the substrate

Apply neat MBrace saturant material to the primed substrate or to the MBrace CF 130 tow sheet as a prime coat using a roller, 9.5 mm nap recommended. Thickness should be approximately 500 micrometers. The color of the MBrace saturant material should be translucent blue.

I. Applying the laminate to the prepared substrate

The CFRP sheet is placed fiber side down onto the concrete surface onto which the wet saturant coat has been applied. After smoothing down by hand, the backing paper is peeled away. The surface of adherent CFRP Sheet must be squeezed in the fiber longitudinal direction using a deforming roller and rubber spatula in order to impregnate resin into the CFRP Sheet and to defoam the resin coat. At the overlapping location, additional resin shall be applied to the outer surface of the Carbon fiber sheet layer to be overlapped.

The CFRP sheet shall have a minimum of 30 minutes between application of sheet into first coat of wet saturant on the concrete and the application of the second coat. This is to allow epoxy impregnation. After 30 minutes apply a topcoat of MBrace saturant resin using a roller, 9.5 mm nap. This topcoat must be applied in fiber longitudinal direction, in order to impregnate and replenish resin into CFRP sheet using a roller in the same film thickness as detailed in item H above. The black tow sheet should be visible. The color of the application will vary due to overlaps and slight thickness variations. Overlap areas should also be translucent.

Minimize the elapsed time between mixing and application of the saturant to ensure the material is applied to the sheet at least 15 minutes prior to any thickening of gelling.

In case more than one layer of CFRP sheet must be laminated, the processes as detailed in sections H through I must be repeated. MBrace Topcoat and MBrace Topcoat ATX shall be used when the system will be subjected to sever and light duty environmental conditions. Brace Topcoat FRP shall be used for fire/smoke protection.

The work must be protected from rain, sand, dust, etc. by using protective sheeting and other barriers. Curing of adherent CFRP must be for no less than 24 hours.

J. Repair of Defective work

Repair of all the defective work (delamination areas) shall be done after the minimum cure time for the CFRP laminates. Comply with material and procedural requirements defined in this specification. Repair all defects in a manner that will restore the system to the designed level of quality. The Engineer shall approve repair procedures for conditions that are not specifically addressed in this specification. All repairs and touch up shall be made to the satisfaction of the Engineer.

K. Testing of the Installed CFRP Laminates

Test all the repaired areas to check for voids, bubbles and delaminations. Repair all voids, bubbles and delaminations by approved methods per Manufacturer's direction.

L. Quality Control and Inspection

1. A qualified representative shall observe all aspects of onsite material preparation and application, including surface preparation, resin component mixing, application of primer, resin and CFRP Sheet, curing of composite, and the application of protective coatings.

2. Inspection for Voids/Delaminations. After allowing at least 24 hours for initial resin cure to occur, perform a visual and acoustic tap test inspection of the layered surface. Large delamination (larger than 1300 mm^2) shall be marked for repair. Voids between 1300 to 16,00 mm^2 shall be repaired by resin injection. For larger voids, the damaged reinforcement area shall be cut around the perimeter and peeled off the surface. The surface shall be smoothed with MBrace putty, primed, and additional MBrace plies shall be applied (with 100 mm overlap) onto the surface and cured in place.

3. Report. The representative shall submit report to the Engineer. Contents shall include both in-process and final inspection details. Contents (as suggested in a private communication with the manufacturer's representative) shall include but may not be limited to:

- Ambient conditions at application.
- Tap test-scanning nap.
- Number and directions of plies applied.
- Cured resin cups.
- Laminated/cured FRP panels.
- Results of tension and pull off tests
- Surface hardness readings.
- Laminate thickness

MEASUREMENTS AND PAYMENT METHODS

Payment for the CFRP sheets on concrete beams will be based on the area of concrete surfaces covered by the sheet and shall include all costs to furnish and install all layers of the sheet, including materials and labor. The cost of preparing the concrete surface for application of the FRP wrap is included in the payment. Individual layers of the sheet will not be paid for separately.

Item	Pay Unit
Carbon FRP Sheet	Square meter

Chipping and patching of deteriorated beams shall be paid for separately.