DISTRICT 3-0 INVESTIGATION OF FIBER-WRAP TECHNOLOGY FOR BRIDGE REPAIR AND REHABILITATION (Phase-I)

Final Report

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SUMMARY

Concrete structures, and particularly concrete highway bridges are exposed over time to the deleterious effects of environmental attacks, leading to chemical degradation of the material due to carbonation and chloride contamination that eventually break the alkali barrier in the cement matrix, and the steel reinforcement in the concrete becomes susceptible to corrosion. As a consequence, the concrete may delaminate at the reinforcement level, leading to cracking and spalling of the concrete due to volume increase of the steel reinforcement. Such degradation is exacerbated by the application of deicing salts on highway bridges, and the freeze-thaw and dry-wet cyclic exposures causing accelerated ageing of the structure over time. In the United States, over 30% of bridges are in need of repair or replacement because of concrete deterioration and consequential steel corrosion, and the percentage is increasing, according to the Federal Highway Administration. Over 200,000 bridges worth $78 billion are in critical need of repair, and it is estimated that $5.2 billion per year in maintenance costs would merely maintain the status quo. Three trillion dollars is needed for rehabilitation/retrofitting of U.S. infrastructure. Similarly in the United Kingdom over 10,000 concrete bridges are in need of structural strengthening. In Europe, the cost of the repair of reinforced concrete structures because of steel corrosion is estimated to be over $600 million annually. In Canada, it is estimated that the required repair costs for parking garages alone is about $6 billion. The concrete deterioration worldwide has motivated the development of new and innovative materials and methods for structural rehabilitation, since replacement of structures would be very costly and nearly prohibited. Externally bonded Fiber-Reinforced Polymer or Plastic (FRP) composites can provide effective solutions for repair and retrofit of aging bridges to extend their intended lifespans.

FRP composite materials in the form of fabrics and laminates have been externally bonded to concrete structures to increase structural capacity and provide longer service-life. Over the last two decades significant research and development work has been

achieved, leading to technology implementation and successful field demonstration projects. FRP materials can act as a permanent "Band-Aid" that would not require large amounts of demolition work to be done before repair begins. FRP benefits of light weight and high strength also make it attractive for strengthening existing concrete bridge structures. FRP can be wrapped like wallpaper around bridge columns and beams to provide additional reinforcement to increase earthquake resistance, durability, and corrosion resistance. Moreover, FRP plates and sheets are easier to install to the surface of a structure, which requires less on-site effort compared to steel plates. FRP composites technology makes repairs very fast and reduces long-term costs.

There is no question that FRP composite materials have shown to be effective for repair and rehabilitation of civil infrastructure. However, there are several potential pitfalls in simply compiling and synthesizing available information without a proper global understanding of the evolution of FRP technologies in construction applications, resulting oftentimes in reported results that are either narrowly focused, or incoherent, or even contradictory. Therefore, it would be highly advantageous to provide a multidisciplinary and comprehensive review, analysis, and synthesis of the current state of knowledge and practice of FRP technology, so that proper and useful recommendations and guidelines could be provided as bases for code developments and rapid practical advancement of the technology. It is in this sense that this project can contribute enormously to effectively translating externally bonded FRP technology into practice. The suitability of the application of this technology, termed "fiber-wrap" by PennDOT, would be of great benefit to PennDOT District 3 in particular and the state of Pennsylvania in general.

This project was undertaken to extensively investigate and evaluate the technical and economical effectiveness of surface bonded FRP technology for concrete bridges in PennDOT's District 3, who is in charge of maintaining approximately 2900 bridges over nine counties, with over 300 T-beam concrete bridges built during the early- mid-1900s which are in need of repair and retrofit. This study includes seven tasks, namely Task I-A: Information Gathering, Task I-B: Analysis of Findings, Task I-C: Defining "Ideal" Candidate Bridge for Repair, Task I-D: Outcomes Measures, Task I-E: Draft Final Report and Oral Presentation, Task I-F: Implementation Strategy, and Task I-G: Final Report and Ideas Have Consequences. This report includes the tasks listed above, as summarized next. Task I-A consists of information gathering through a comprehensive literature review and a survey/interview of various relevant information pertaining to strengthening of concrete members with external FRPs. This includes FRP constituent materials and durability; concrete materials and deterioration mechanisms; applications, flexural and shear response of external FRP reinforcements for concrete structures; FRP-concrete bond interface performance; durability studies; project case histories; and surveys on applications, implementation, evaluations and assessments of FRP technology. Task I-B consists of assessing the data gathered in Task I-A. This includes applicable situations, implementation protocols, effectiveness, life-span and durability, cost-effectiveness, and limitations of FRP technology. Task I-C consists of defining an ideal candidate bridge in PennDOT District-3's inventory for suitability of repair with externally bonded FRP composites. This includes criteria for classification of bridges based on age, span, ADT/ADTT, and extent of damage from photographs and field visitations of selected 16 bridges. Task I-D focuses on developing outcomes measures, including a comparison of the process and cost-effectiveness of the two repair methods, namely conventional rehabilitation technique and repair using FRP technique. Furthermore, a comparison between replacement and complete rehabilitation using FRP technology is provided. Task I-F provides an implementation strategy, including identification of candidate bridges, field assessment of the structure, evaluation of in-situ material properties from field samples, structural analysis for existing conditions, design of FRP repair, repair implementation and evaluations, testing of repaired structure, and supporting lab-scale studies. Finally, conclusions and recommendations are provided.

Section 1 – Task I.A: Information Gathering

This task includes two sub-tasks: Literature Search (I-A.1) and Surveys and/or Interviews (I-A.2). The literature search provides a comprehensive report with details of relevant information pertaining to strengthening of concrete members with FRP. The survey/interview section describes the mechanism used and results obtained by contacting state and federal highway transportation officials, specialized contractors, suppliers, and design engineers, who have had experiences with this technology.

Sub-Task I-A.1: Literature Search

This section provides information on surface-bonded FRP repair technology. The literature search provides a comprehensive review on all relevant information pertaining to retrofitting of concrete members with FRP. The information includes: (1) FRP constituent materials and durability; (2) concrete materials and deterioration mechanisms; (3) external FRP reinforcements for concrete structures, including information and applications, flexural and shear responses, FRP-concrete bond interface performance, durability studies, and project case histories.

I-A.1.1 FRP Materials and Durability

Fiber-reinforced polymer or plastic (FRP) composites have favorable properties in relation to conventional construction materials. They consist of high-strength fiber systems embedded in a polymer matrix and tailored for specific structural requirements. In infrastructure applications, and particularly for strengthening of concrete, several fiber types (glass, carbon, and aramid) and sometimes hybrid combinations (carbon/glass, aramid/glass) with distinct structural and durability characteristics are available. Similarly, thermoset resins typically used in infrastructure are available in different formulations, and their compatibility characteristics are of paramount importance; the resin or matrix imparts transverse properties to the composite (shear, tension), protects the fibers (impact, fire), and being in direct contact with the exterior environment, its durability under harsh and cyclic weathering conditions is of great concern and consequence to structural performance in construction. For example, in an effort to enhance performance, polyurethane resins have been recently developed for production of structural shapes by pultrusion. For external reinforcement of concrete with fabrics, the resin serves the dual role as matrix for the fibers and simultaneously as adhesive for the bi-material system. Thus, a general knowledge of fibers and resins and their mechanical and environmental characteristics is necessary, and it is therefore part of the present literature review.

I-A.1.1.1 Resin/Matrix

The resins have three important roles as matrix for fibers: to transfer stress of fibers to other fibers, to hold fibers together, and to protect fibers from the environmental and mechanical damage. There are two main types of polymers used for resins: thermosets and thermoplastics. Table I.A.1.1 shows the typical properties of matrices for FRP.

Table I.A.1.1 Typical Properties of Matrices Used in Fiber Reinforced Plastic Composites. (ACI 440, 2002; and Barbero, 1999)

Matrix	Density (g/cm^3)	Tensile modulus (GPa)	Tensile strength (MPa)	Tensile Elongation (%)	Tg^*(C)
Thermoset:					
Polyesters	1.1-1.4	2.8-3.4	20.7-75.9	1.4-3.3	-
Vinyl Esters	1.12	3.4	82.7	5-6	-
Epoxies	1.16-1.2	1.4-3.38	26.2-77.2	3.1-5.2	155-239
Thermoplastic:					
PEEK	1.32	3.24	100	50	143
PPS	1.36	3.3	82.7	5	90
PEI	1.27	3	105	60	217
PAI	1.4	2.76	89.57	30	243

$^*T_g$ (glass transition temperature): the approximate midpoint of the temperature range over which the glass transition takes place. The glass transition means the reversible change in an amorphous polymer, or in amorphous regions of partially crystalline polymer, from (or to) a viscous or rubbery condition to (or from) a hard and relatively brittle one.

***Thermosetting matrix*:** Thermosetting matrix is formed by irreversible chemical transformation of a resin into an amorphous cross-linked polymer matrix. Thermosetting resins are the most commonly used in civil engineering applications because of their favorable properties such as low cost, ease of processing (low viscous), good thermal stability and chemical resistance. Polyester, vinyl esters and epoxies are the most popular thermosetting matrices.

***Thermoplastic matrix*:** A thermoplastic polymer is softened from solid state to be processed, and returns to solid state after being processed. Compared with the thermosetting resins, it does not undergo any chemical transformation during processing. Therefore, thermoplastic matrix can be repaired by being transited to soft stage. Thermoplastics have unlimited shelf life, and they are much more viscous than thermosettings. Polyester ether ketone (PEEK) is the most commonly used thermoplastic matrix. Other thermoplastics such as polyphenylene sulfide (PPS), Polyetherimide (PEI) and polyamide-imide (PAI) are also available.

For structural applications it is mandatory to achieve some degree of flame retardance. Fire retardants are usually incorporated in the resin itself or as an applied gel-coat. Fillers and pigments are also used in resins for a variety of purposes, the former principally to improve mechanical properties and the latter for appearance and protective action.

I-A.1.1.2 Fibers

A wide range of amorphous and crystalline materials can be used as the fiber (Network Group for Composites in Construction 2004). In the construction industry the most common fiber used is glass fiber. Carbon fiber can be used separately or in conjunction with glass fibers as a hybrid to increase stiffness and possibly strength of a structural member. Aramid fibers can be used instead of glass fibers to give increased stiffness to the composite. In addition to the fibers mentioned above, there are some other fibers such as polyvinyl alcohol fibers, boron fibers and organic fiber spectra produced by Allied Signal Corp. in the USA. Table I.A.1.2 shows typical properties of fibers.

Bundles of filaments are called strands and these are usually combined to form thicker parallel bundles called rovings. Assembled rovings are used in processes involving chopping of the fibers during the production of a composite e.g. SMC, spray-up, continuous sheet manufacture. Rovings are also manufactured by a direct technique in which all the filaments needed in the final roving (up to 4800) are all drawn simultaneously from one bushing. These rovings are called direct rovings and are used in weaving, pultrusion and filament winding. Strands may also be twisted to form several types of yarn; rovings or yarns may be used either individually or in the form of a woven fabric.

Table I.A.1.2 Typical Properties of Fibers Used in Fiber Reinforced Plastic Composites. (ACI 440, 2002; and Uomoto, 2002)

Fiber	Diameter (μm)	Density (g/cm^3)	Tensile modulus (GPa)	Tensile strength (GPa)	Elongation at break (%)
Glass:					
E-glass	10	2.54	72.4	3.45	4.8
S-glass	10	2.49	86.9	4.30	5.0
AR-glass	8-12	2.27	68.6-74.5	1.8-3.4	2-3
Carbon:					
PAN-Carbon T-300	7	1.76	231	3.65	1.4
PITCH-Carbon P-555	10	2.0	380	1.90	0.5
Aramid:					
Kelvar 49	11.9	1.45	131	3.62	2.8
Twaron 1055	12.0	1.45	127	3.6	2.5
Technora	12	1.39	72.5	3.43	4.6

Glass fibers**:** E-glass fibers are made of calcium-alumina-silicate glass and comprise about 80 to 90 percent of glass fiber commercial production. The boron-free modified E-glass is named as ECR-glass, which offers improved resistance to acid corrosions (ACI 440, 2002). S-glass fibers (S for strength) are proprietary magnesium alumino-silicate formulations. S-glass with higher strength are three to four times more expensive than E-glass. S-glass and S-2 glass have same composition but different surface treatments. Alkali resistant (AR) glass fibers are also available. They contain a large amount of zirconia (ZrO_2), which could potentially prevent corrosion by alkali attacks. There are also other glass fibers such as C-glass (C for corrosion) and D-glass (D for dielectric). Generally, glass-fibers exhibit good electrical and thermal insulation and magnetic neutrality, but lower elastic modulus than steel. Due to economic advantages, glass fibers have been predominantly used in civil engineering applications.

Carbon fibers**:** Carbon fibers are made from pitch, petroleum and polyacrylonitrile (PAN). The fiber, an aggregate of graphite crystals, is also called graphite fiber. The properties depend on the material molecular structure and defects. The pitch carbon and PAN carbon fibers are two commercially available fiber types. Carbon fibers offer an excellent combination of strength, high modulus and low weight, but they are more brittle and expensive than glass and aramid fibers.

***Aramid fibers*:** Aramid fiber is a kind of organic fiber, which is poly-para-phenyleneterephthalamide (PPD-T). There are three commercially available aramid fibers: Kevlar^{TM}, Twaron^{TM} and Technora^{TM}. Aramid fibers have good electrical and heat insulation, and are resistant to fuels, lubricants and organic solvents. Compared to glass and carbon fibers, aramid fibers have reasonable high tensile strength, a medium modulus, and a very low density. Aramid composites have good impact resistance but lower compressive strength than glass and carbon composites.

I-A.1.1.3 Mechanism of Reinforcement

The reinforcement of a low modulus polymer with a high modulus, high strength fiber uses the plastic flow of the polymeric material under stress to transfer the load to the fiber; this results in a high strength, high modulus composite. The aim of the combination is to produce a two-phase material in which the primary phase (i.e. the fibers) is well dispersed and bonded by a weak secondary phase (i.e. the polymer matrix). The principal constituents influencing the strength and stiffness of composites are the reinforcing fibers, the matrix and the interface between the fibers and the matrix.

I-A.1.1.4 History of FRPs in Construction

Fiber reinforced polymer (FRP) composites were first developed during the 1940's, for military and aerospace applications. Considerable advances have been made since then in the use of this material and applications developed in the construction sector. FRPs have been successfully used in many construction applications including load bearing and infill panels, pressure pipes, tank liners, roofs, and complete structures where FRP units are connected together to form a complete system, in which both the shape and material lay-up provide the desired rigidity.

In the last decade, polymer composites have found application in the construction sector in areas such as bridge repair, bridge design, mooring cables, structural strengthening and stand-alone components. These composites are materials often referred to as ***advanced composites*** and have properties considerably superior to those of earlier composites. The term is ambiguous, however, because it does not identify any specific material combination. In the construction industry, the term is generally used for polymers reinforced with high strength and high modulus continuous fibers of glass, carbon or aramid laid up in layers to form an engineered material.

I-A.1.1.5 Durability of Composites

In this section, a relatively comprehensive review on durability of composites relevant to construction is summarized. The review includes issues such as fiber-matrix interactions (voids, fiber-surface coatings) in the presence of moisture, exposure to moisture and temperature, low temperature effects, sustained exposure to water and salt solutions, and particularly wet/dry cycling in salt solutions, and freeze-thaw in water and salt, and other effects such as ultraviolet exposure.

Thomason (1995) found that the fiber-matrix interface has a strong influence on the properties of glass fiber-reinforced epoxy matrix composites. Interfacial strength, as measured by the interlaminar shear strength (ILSS) varied considerably depending on the nature of the fiber surface coating and on the type of curing agent in the resin formulation. Removing this surface coating by first heating the fibers to 500^{o}C led to a dramatic decrease in the composite interface strength. The fiber surface coating was also found to play a vital role in the retention of interfacial strength in a wet environment. The void content of the composite was found to have a strong influence on the ILSS. The magnitude of this dependence varied in the range 2-10 MPa loss per 1% voids, depending on the fiber/matrix combination. The void, the fiber surface coating and the nature of the fiber-matrix interface influence water absorption in glass fiber-reinforced epoxy matrix composites. The presence of only 1% voids in E-glass fiber-reinforced EPIKOTE resin composites can more than double the amount of water it absorbs. A surprising result was that removal of the fiber surface coating had no apparent effect on the kinetics of water absorption, despite the large detrimental effect on the interfacial strength.

The relationship between hygrothermal histories and durability of epoxy resins and their glass bead composites has been studied by Apicella et al. (1982). They showed that sorbed moisture deteriorates the mechanical integrity of the epoxy matrix, in a manner which is dependent upon the temperature and humidity to which the material has been previously exposed. Aging in hot water has been described to raise T_g in epoxy resins due to the activation played by the water molecules. Thereafter, they performed a mechanical and calorimetric analysis on a glass fiber-reinforced polyester resin aged in water at different temperatures. The reported elastic moduli for the 'as prepared' and the thermally aged in water at 90^oC for 15 days, showed an increase for both the pure resin and the composite. However, the elongations at break showed a strong reduction for both the pure resin and the composite, from 1.7 to 0.5 and 1.4 to 0.4. They suggested that there is a competitive effect between matrix plasticization as a result of water sorption and stiffness increase due to the loss of low molecular weight substance.

The effects of moisture and temperature on graphite/epoxy specimens were investigated by Demuts and Shyprykevich (1984). The fiber dominated tensile strength is not significantly affected by the moisture or the elevated temperature. The matrix dominated compressive strength, however, is reduced significantly by increased moisture and elevated temperature, but is not significantly affected by moisture conditioning method or spectrum loading.

Low temperature produces internal stresses in composites of polymeric materials. The polymeric matrix phase becomes stiffer, and may suffer from damage-inducing stresses resulting from thermal coefficient mismatch of fibers and resins. These effects have been studied by Dutta and Hui (1996). The results of the three-point bending test for both S2-glass composite and E-glass composite showed that both the Young's modulus (E) and shear modulus (G) values increase with reduction of temperature. For S2-glass composites, the E value increased approximately at a rate of 10.5 MPa/^oC (846 psi/^oF), and G at 2.41 MPa/^{o}C (194 psi/^{o}F). The increase of E for E-glass composite is more dramatic, 72.1 MPa/^{o}C (5806 psi/^{o}F). The increase of E values in both composites was primarily the result of increase of the E value of the matrix at low temperatures. The low temperature thermal cycling has a significant influence on the degradation of both the E and G values of the composite. The Young's modulus degraded 6.2% and shear modulus degraded 6.3%. The degradation is primarily the result of matrix degradation.

In general, at low temperature the FRP composites will stiffen up. The performance of its matrix dominated behavior will improve. The increase of E and G values at lower temperatures control the composite's flexural properties. The polyester resin matrix of E-glass composites appears to have a higher rate of modulus increase than the polyester matrix composite used with S2-glass. Low temperature thermal cycling has shown degradation of both the Young's modulus and shear modulus of the plain-weave glass composites.

Pultruded GFRP coupons aged in several different conditions were studied by Liao et al. (1999). Both strengths and moduli of pultruded E-glass fiber-reinforced vinyl ester composite were generally found to decrease with environmental aging in water or salt solutions at room temperature (25^{o}C) or in water at 75^{o}C for various times. The degradation of the fiber/matrix interphase region also occurred during the aging process.
The flexural modulus of 0^{o} specimens aged in de-ionized water for up to 3900 h at room temperature (25^{o}C), or 5% salt solution for up to 3980 h at room temperature, or 10% salt solution for up to 6570 h at room temperature, remained essentially the same. The flexural strength after aging, however, showed some degradation. The failure strains also decreased for the aged specimens. A more significant drop in flexural strength was seen in specimens aged in de-ionized water for 2400 h at 75^{o}C where an average 40% drop in flexural strength was seen. The failure strain was 1.34%. For 90^{o}C specimens, compared to un-aged specimens, the flexural modulus for coupons aged in room temperature water for 7940 h and 75^{o}C water for 1360 h showed 13% and 19% drop, respectively. Significant flexural strength degradation was seen for the 90^{o}C specimens.

In general, the higher the temperature of the environment and the longer the exposure time, the larger decrease in strength and modulus of GFRP. Compared to aging at lower temperature, significant degradation for GFRP occurs at temperatures above 70^{o}C, a consequence of fiber/matrix debonding leading to delamination and cracking combined with plasticization of the matrix.

The durability of advanced polymer composites exposed to wet/dry cycling in water and salt solutions, and UV radiation was investigated by Hulatt et al. (2002). Exposure of prepregs of CFRP and GFRP to a wet/dry cycle incorporating solutions of tap water and a saturated salt solution (road salt, sodium chloride) produced no noticeable adverse effects on the longitudinal modulus. Slight increases were shown for all composites materials, apart from the 0/90^{o} GFRP in salt solution which showed only a 3% reduction in modulus. The ultimate failure stress was reduced by exposure to tap water for the materials with fibers that lay in the line of action of the load, but there was an increase in this stress with exposure to the salt solution environment. The water or salt solution environments had no adverse effect on the ultimate strength of the matrix-dominated +/-45^{o} GFRP. No adverse effects on the mechanical properties were found in specimens subjected to UV radiation for 2000 h. Slight reductions in the modulus were found in the GFRP composites, but a slight increase was found for the CFRP. Discoloration of the matrix material was evident, particularly in respect to the GFRP composites.

Freeze-thaw durability of composites in water and salt water was studied by Haramis et al. (2001) and Rivera et al. (2002). Strength for the toughened vinyl ester was 389 MPa in the as-received condition versus 237 MPa for the post-saturation condition. Likewise, for the untoughened vinyl ester, strengths were 432 MPa versus 240 MPa. For the epoxy, strengths were 424 MPa versus 237 MPa. Stiffness for the toughened vinyl ester was 19.9 GPa in the as-received state versus 22.1 GPa for the post-saturation condition. Stiffness for the untoughened vinyl ester was 23.9 GPa for both conditions. For the epoxy, stiffness was 26.2 GPa versus 25.6 GPa. Strain-to-failure for the toughened vinyl ester was 2.49% in the as-received condition versus 1.26%% for the post-saturation condition. For the untoughened vinyl ester, strain-to-failure was 2.58% versus 1.36%, and for the epoxy, 2.29% versus 1.20%. In summary, strength and strain-to-failure were approximately 50% lower after saturation, but stiffness effectively remained unchanged for the toughened vinyl ester, the untoughened vinyl ester, and the epoxy samples (Haramis et al. 2001).

The short-term exposure of thin ambient cured carbon/vinyl ester specimens to freeze and freeze-thaw cycling was investigated by Rivera and Karbhari (2002). It was found that freeze-thaw can cause significant reduction in mechanical properties and in glass-transition temperature with immersion in salt water, having a larger effect on fiber-matrix bond deterioration and matrix cracking than other exposure. All of the freeze-thaw exposures resulted in drops in tensile strength and modulus as a consequence of fiber-matrix debonding and matrix microcracking. The effects on modulus were significantly smaller than those on strength. Although there was an indiscernible difference in modulus change as a result of freeze-thaw in water and salt-water, there was a clearly noticeable increase in strength loss due to the salt-water environment. Exposure to -10^{o}C resulted in an increase in compressive strength due to matrix hardening.

Durability and environmental degradation of glass-vinylester composites exposed to high temperature, moisture, seawater, and corrosive fluid were evaluated by Hammami and Al-Ghuilani (2004). They concluded that immersing in seawater will lead to poor performance of specimens, and the interlaminar shear strength can decrease significantly. For corrosive fluids, by increasing the medium concentration and immersion time, the performance of the composite specimens will change drastically. The combined action of water and corrosive fluid will lead to matrix expansion and the occurrence of pits. In their study, specimens exposed to high temperatures and a fully saturated environment were subjected to degradation caused by water diffusion through the matrix. Specimens exposed to high temperatures and a completely dry environment showed a decrease in the mechanical properties after a three-month period; while for a six-month period, values recorded for the mechanical properties were higher than those recorded with the virgin specimens.

The effect of seawater immersion on the durability properties of conventional marine composite materials made of glass/polyester and glass/vinyl ester against a new generation of marine composites made from carbon/polyester and carbon/vinyl ester, was first studied by Kootsookos and Mouritz (2004). The study showed that polyester-based composites are less chemically stable in seawater than vinyl ester-based composites. Despite the superior chemical stability, the flexural properties of the vinyl ester-based composites were degraded to a similar extent to the properties of the polyester materials. It was found that fiberglass composites absorb more moisture than carbon fiber composites. The mode I interlaminar fracture toughness of the composites was not affected significantly by seawater immersion, although the flexural stiffness and strength decreased with increasing amounts of water absorption.

As can be seen from the above literature review, the results of the durability studies are not consistent. Some studies such as those conducted by Apicelli et al. (1982) and Dutta et al. (1996) showed an increase in strength with aging, whereas the results of studies conducted by Demuts et al. (1984), Liao et al. (1999), Haramis et al. (2001), and Rivera et al. (2002) showed reduction in the strength with increased aging. This inconsistency in the results could be attributed to the lack of standard environmental aging protocol and test methods, to evaluate the performance of composites subjected to harsh weathering cycles.

The above results and some other reviews are concisely described in Table I.A.1.3.

Table I.A.1.3 Summarized Reviews on FRP Durability.

No.	Title	Author / Authors	Source	Composite Types	Exposure Conditions	Selected Outcome/Conclusions
1	Interface region in glass fibre-reinforced epoxy resin composites: 3. Characterization of fibre surface coatings and the interphase	J.L.Thomason	Composites, v 26, n 7, 1995, p 487-498	Glass/ epoxy	Moisture	Interlaminar shear strength (ILSS) depends on the fiber surface coating and the void content. The void, the fiber surface coating and the nature of the fiber-matrix interface influence water absorption.
2	Water sorption and mechanical properties of a glass-reinforced polyester resin	A. Apicella, C. Migliaresi, L. Nicolais, L. Iaccarino, S. Roccotelli	Composites, v 13, n 4, 1982, p 406-410	Glass/ polyester	Water at different temperature	Elastic moduli for the 'as prepared' and the thermally aged in water at $90^{o}C$ for 15 days, showed an increase for both the pure resin and the composite. The elongations at break reduced for both the pure resin and the composite.
3	Accelerated environmental testing of composites	E. Demuts, P. Syprykevich	Journal of Composites, v15, n 1, 1984, p 25-31	Graphite/ epoxy	Moisture and temperature	Tensile strength is not significantly affected by the moisture or the elevated temperature. Compressive strength is reduced significantly by increased moisture and elevated temperature.
4	Low-temperature and freeze-thaw durability of thick composites	P.K.Dutta, D.Hui	Composites Part B: Engineering, v 27, n 3-4, 1996, p 371-379	E-glass/ Polyester and S-glass/ polyester	Low temperature and freeze-thaw cycling ($50^{o}C$ to $-76^{o}C$)	Young's modulus (E) and shear modulus (G) increase with reduction of temperature. The increase of E primarily is as a result of increase of the E value of the matrix at low temperatures. The low temperature thermal cycling reduces both the E and G values of the composite, but the effect is primarily the result of matrix degradation.
5	Freeze-thaw durability of composites for civil infrastructure	J.Haramis, K.N.E. Verghese, J.J. Lesko	Plastics Failure Analysis and Prevention, 2001, p 113-120	Toughened vinyl ester, untoughened vinyl ester, epoxy	Freeze-thaw cycling	Strength and strain-to-failure were approximately 50% lower after saturation, but stiffness effectively remained unchanged for the three resins.

No.	Title	Author / Authors	Source	Composite Types	Exposure Conditions	Selected Outcome/Conclusions
6	Effects of environmental aging on the properties of pultruded GFRP	K.Liao, C.R. Schultheisz, D.L.Hunston	Composites Part B: Engineering, v 30, 1999, p 485-493	E-glass/ vinyl ester	Water or salt solutions at $25^{o}C$ or at $75^{o}C$	Both strengths and moduli decrease with environmental aging in water or salt solutions at room temperature ($25^{o}C$) or in water at $75^{o}C$ for various times. The failure strains also decreased for the aged specimens. In general, the higher the temperature of the environment and the longer the exposure time, the larger decrease in strength and modulus of GFRP.
7	Preliminary investigations on the environmental effects on new heavyweight fabrics for use in civil engineering	J.Hulatt, L.Hollaway, A. Thorne	Composites Part B: Engineering, v 33, 2002, p 407-414	Carbon fiber/epoxy, glass fiber/epoxy	Three temperatures (22, 45, and $60^{o}C$), water, salt solution, ultra-violet (UV)	Exposure of CFRP and GFRP to a wet/dry cycle incorporating solutions of water and a salt solution produced no noticeable adverse effects on the longitudinal modulus. No adverse effects on the mechanical properties were found in specimens subjected to UV radiation for 2000 h.
8	Cold-temperature and simultaneous aqueous environment related degradation of carbon/vinylester composites	J.Rivera, V.M. Karbhari	Composites Part B: Engineering, v 33, n 1, 2002, p 17-24	Carbon/ vinylester	Freeze and freeze-thaw cycling	The freeze-thaw exposure all resulted in drops in tensile strength and modulus. The effects on modulus are significantly smaller than those on strength. Exposure to $-10^{o}C$ results in an increase in compressive strength due to matrix hardening.
9	Seawater durability of glass- and carbon-polymer composites	A.Kootsookos, A.P. Mouritz	Composites Science and Technology, v 64, 2004, p 1503–1511	Glass/ polyester, carbon/ polyester, glass/ vinylester, carbon/ vinylester	Immersed in seawater at a temperature of $30^{o}C$ for over two years	Composites experienced significant moisture absorption and suffered chemical degradation of the resin matrix and fiber/matrix interphase region. Flexural modulus and strength of the composites degraded. The mode I interlaminar fracture toughness was only marginally affected by immersion.

No.	Title	Author / Authors	Source	Composite Types	Exposure Conditions	Selected Outcome/Conclusions
10	Durability and environmental degradation of glass-vinylester composites	A.Hammami, N.Al-Ghuilani	Polymer Composites, v 25, n 6, 2004, p 609-616	Glass/ vinylester	High temperature, moisture, seawater, corrosive fluid	Seawater degrades performance of the specimens and makes the interlaminar shear strength decrease significantly. The combined action of water and the corrosive fluid leads to matrix expansion and the occurrence of pits. High temperatures and a fully saturated environment cause degradation of specimens. High temperatures and a completely dry environment decrease mechanical properties after the three-month period, while increasing mechanical properties for the six-month period compared with the virgin specimens.
11	Prediction of deterioration of FRP rods due to alkali attack	Katsuki F & Uomoto T	In: Taerwe L (ed) Non-metallic (FRP) reinforcement for concrete structures: Proceedings of the Second International RILEM Symposium (FRPRCS-2), Ghent, Belgium, 23d25 August 1995. London: E & FN Spon, 1995: 82d89.	Glass, aramid & carbon FRP	NaOH solution	The alkali penetrated the GFRP rods radially with time while the CFRP and AFRP rods had no penetration of alkali. Only the GFRP rods lost strength with time after exposure to alkali. The area of the GFRP rods penetrated by alkali failed at a lower load than the areas not penetrated by the solution.
12	Creep-rupture of fiber-reinforced plastics in a concrete environment	Dolan CW, Leu BL & Hundley A.	In: Non-metallic (FRP) reinforcement for concrete structures: Proceedings of the Third International Symposium. Sapporo, Japan, 14-16, October 1997. Vol. 2. Tokyo: Japan Concrete Institute. 1997:187-194.	Glass, aramid & carbon fibers	Creep rupture	Results from the short-term tests showed that there was substantial decay in the glass tendons while there was no decay in the aramid and carbon tendons.

I-A.1.2 Concrete Materials and Deterioration Mechanisms

Concrete has become the world's most widely used construction material (340 million cubic yards annually of ready-mix concrete in the US) because of its versatility and cost-competitiveness. There are however deteriorating mechanisms in concrete that need to be understood for proper evaluation of performance and applications of repair and retrofitting methods. Major deteriorating mechanisms in concrete (Neville 1996) include alkali-silica reaction from potential silica content in the aggregate, freeze-thaw damage usually due to critically saturated concrete, delayed ettringite formation related to some form of internal sulfate attack, external acid and sulfate attacks, and carbonation which can lower the pH of the paste to about 7.0 leading to exacerbated corrosion in reinforced concrete. Of major concern with concrete in highway bridges is the freeze-thaw effect, which for normal concrete was shown to cause major damage for a critical temperature range of 0^o to -10^o C.

Sulfate Attack: When solid salts present in solution, they can react with hydrated cement paste. Sulfates of sodium, potassium, magnesium, and calcium occur naturally in soil or groundwater. Sulfates in groundwater can also arise from the use of fertilizers or from industrial effluents. These sometimes contain ammonium sulfate, which attacks hydrated cement paste by producing gypsum ($CaSO_4.2H_2O$). Soil in some disused industrial sites, particularly gas works, may contain sulfates and often other aggressive substances. Reactions of sulfates with hardened cement paste include sodium sulfate attacks the $Ca(OH)_2$, calcium sulfate attacks the calcium aluminate hydrate ($3CaO.Al_2O_3.12H_2O$) forming calcium sulfoaluminate ($3CaO.Al_2O_3.3CaSO_4.32H_2O$) which is known as ettringite, and magnesium sulfate attacks the calcium silicate hydrates in addition to $Ca(OH)_2$ and the calcium aluminate hydrate (Neville 1996).

In addition to disruptive expansion and cracking by sulfate attack, there is also a loss of strength of concrete due to the loss of cohesion in the hydrated cement paste and of adhesion between it and the aggregate particles. Concrete attacked by sulfates characteristically has a whitish appearance termed as efflorescence, with the damage usually beginning at edges and corners followed by progressive cracking and spalling.

Alkali Attack: The porewater environment of ordinary Portland cement concrete is one of high humidity and pH – in the range of 10-13.5 (Andersson K. et al., 1989). The composition of dry ordinary Portland cement primarily consists of calcium and silicon oxides (CaO and SiO_2) – also known as lime and silica, respectively – with smaller quantities of oxides of aluminum, iron, magnesium and potassium. The two calcium silicates ($3CaO.SiO_2$ and $2CaO.SiO_2$), which constitute about 75% of the weight of Portland cement, react with water in a process known as hydration to form two new compounds: calcium hydroxide and calcium silicate hydrate (CSH). During hydration, the cement hardens into its gel phase – the cohesive mass of hydrated cement in its densest paste-state – with byproducts of calcium hydroxide ($Ca(OH)_2$), sodium hydroxide ($NaOH$), and potassium hydroxide (KOH). Most of these hydroxides remain mixed with the concrete porewater and contribute to the high pH of the resulting solution (Diamond, 1981).

Acid Attack (Neville 1996): Concrete is susceptible to acid attack because of its alkaline nature. The components of the cement paste break down during contact with acids. The decomposition of the concrete depends on the porosity of the cement paste, concentration of the acid, solubility of the acidic calcium salts (CaX_2), and fluid transport through the concrete. Insoluble calcium salts may precipitate in the voids and can slow down the attack. Acids such as nitric acid, hydrochloric acid and acetic acid are very aggressive as their calcium salts are readily soluble and removed from the attack front. Other acids such as phosphoric acid are less harmful as their low solubility calcium salts inhibit the attack by blocking the pathways within the concrete. Sulfuric acid is very damaging to concrete as it combines an acid and sulfate attack. Absence of calcium hydroxide in the cement paste and surface dissolution of cement paste exposing the aggregates are both signs of acid attack.

Freeze-Thaw Attack: The deterioration for concrete may be created when cold concrete is exposed to warmer, moist air on one side and evaporation is insufficient or restricted on the cold side, or when the concrete is subjected to a head of water for a period of time prior to freezing (ACI 201.2R-92, 1997). As the water in moist concrete freezes, it produces osmotic and hydraulic pressure in the capillaries and pores of the concrete paste and aggregate. If the pressure exceeds the tensile strength of the paste or aggregate, the cavity will dilate and rupture. The accumulative effect of successive freeze-thaw cycles and disruption of paste and aggregate eventually cause significant expansion and deterioration of the concrete. Deterioration is visible in the form of cracking, scaling and crumbling. Hydraulic pressures are caused by the 9% volume-expansion of water that results upon freezing, in which growing ice crystals displace unfrozen water. Osmotic pressures develop from differential concentrations of alkali solutions in the paste.

Freezing of pore solution in concrete exposed to a freeze-thaw cycle was studied by Cai and Liu (1998). Pore solution freezes more quickly above -10^{o}C than below -10^{o}C. Higher freezing rate leads to greater internal hydraulic pressure. Internal hydraulic pressure is a direct factor causing frost deterioration of concrete. So concrete suffers more destructive effects from freezing between 0^{o}C and -10^{o}C whether it is ordinary or high-strength concrete. Damages mainly occur in the range from 0^{o}C to -10^{o}C and have limited increase below -10^{o}C. A possible reason that may explain the phenomenon is that there is a concentrated distribution of solution in pores that have the most probable pore size and solution in these pores freezes above -10^{o}C. They suggested that study of freeze-thaw durability of ordinary concrete should concentrate on the temperature range between 0^{o}C and -10^{o}C.

Delayed Ettringite Formation (DEF) (Neville 1996): Generally DEF is seen as a form of internal sulfate attack. A number of factors such as concrete composition, curing conditions and exposure conditions influence the potential for DEF, which is believed to be a result of improper heat curing of the concrete where the normal ettringite formation is suppressed. The sulfate concentration in the pore liquid is high for an unusually long period of time in the hardened concrete. Eventually, the sulfate reacts with calcium- and aluminum-containing phases of the cement paste and the cement paste expands. Due to this expansion empty cracks (gaps) are formed around aggregates. The cracks may remain empty or later be partly or even completely filled with ettringite. DEF can be detected by: the presence of gaps completely encircling aggregates, wider gaps around larger aggregate, absence of external sulfate source and high temperature heat curing history.

Carbonation (Neville 1996): Carbonation, i.e. the reaction of $Ca(OH)_2$ within the concrete porewater and carbonic acid (H_2CO_3 – stemming from environmental carbon dioxide, CO_2) in forming calcium carbonate, $CaCO_3$, can lead to a significant drop in pore water pH over time. Complete carbonation of a concrete reduces the pH to a level of 9 or lower (Padadakis et al. 1992) leading to exacerbated corrosion in reinforced concrete.

Cement paste contains 25-50 % calcium hydroxide ($Ca(OH)_2$), which means that the pH of the fresh cement paste is at least 12.5. The pH of a fully carbonated paste is about 7. Carbonation results in a decrease of the porosity making the carbonated paste stronger. Carbonation is therefore an advantage in concrete without steel reinforcements. However, it is a disadvantage in reinforced concrete, as pH of carbonated concrete drops to about 7, a value below the passivation threshold of steel. Carbonation may be recognized in the field by the presence of a discolored zone in the surface of the concrete. The color may vary from light gray to strong orange.

The degradation process of concrete simultaneously exposed to loading, freeze-thaw cycles, and chloride salt attack will be significantly accelerated (Mu et al. 2002). The weight loss of concrete specimens is caused by the scaling of concrete surface. In actual concrete applications, concrete surface scaled markedly when exposed to deicing salt and freeze-thaw cycles caused by the change of climate. The weight loss in a NaCl solution was twice as large as that in water. However, the DME (dynamic modulus of elasticity) loss of concrete immersed in a NaCl solution is less than in the water. The number of freeze-thaw cycles at failure in a NaCl solution was roughly 20% higher than those in fresh water.

I-A.1.3 FRP-Concrete: Applications, Structural Response, Bond Interface, Durability and Case Studies

In the last two decades, external reinforcement with FRP materials has proven to be an efficient technology for repair and retrofit (Meier et al. 1993). The term “repair” is used when the structure is restored to its intended functional level, such as in cases of design error, insufficient reinforcement, or section loss due to aging and corrosion; while the term “retrofit” implies functional upgrade, such as for increasing traffic load or adding lanes.

I-A.1.3.1 Applications

FRP composite products have been widely used in the aerospace, electronics, marine and corrosion resistance industries for several decades, but their application in concrete engineering as a reinforcing material is relatively recent in origin (Uomoto 2002). Several developments in the construction industry have accelerated the efforts to apply FRP as a reinforcing material in concrete. One of the chief causes is the realization that the reinforcing steel is corrosion prone. This realization obviously prompts the desire to use noncorrosive materials such as FRP, especially in environments where steel has been shown to be vulnerable. Besides being noncorrosive, the FRPs have a much higher strength-to-weight ratio and this makes them an ideal material for applications in repair, rehabilitation, and strengthening works. In certain special applications, use of a nonmagnetic material such as FRPs has an added advantage.

In addition, FRP strengthening poses a number of potential advantages (Taljsten 2004). Carbon fiber composites have especially good durability, long-term fatigue properties, and do not need to be maintained over time. In many situations, thin strengthening layers can be advantageous. Thin layers will not change the dimension of the existing structure and can also be combined with thin concrete overlays or surface protecting materials. FRP strengthening can often be done during short periods without stopping the traffic, and little time is needed for hardening of the bonding agents. During the last few years, products have been introduced to the market that can be prestressed in combination with bonding. This gives a higher utilization of the strengthening product, at the same time reducing existing cracks, and increasing the yield load of the existing steel reinforcement. It is also possible to use prestressing to increase the shear capacity of concrete structures.

Extensive research across the world during the last 25 years or so has led to a better understanding of the properties and behavior of the FRPs under different conditions, and more extensive use of FRPs is likely to be seen in the coming years (Taerwe et al 2001). The common forms of FRP products used as reinforcement in concrete structures include 1D forms such as bars, tendons and strands, subjected to high levels of tensile load; 2D thin-shell or plate-like elements, such as grids, jackets and tapes; and 3D fabrics. FRP fabrics and plates are the most commonly used as external reinforcement.

FRP products, as a replacement of steel, are bonded to concrete beams and slabs and wrapped around concrete columns. In Japan, carbon FRP sheets have been used to strengthen shear and flexural capacities of structures, such as highway piers due to change in design wheel loads in 1993 (Uomoto 2002). About 200 columns in 1993 and 1994 were wrapped with FRP products in Los Angeles and Santa Monica. Grace et al. (1999) reported a successful field application of CFRP rods for externally strengthening ten pier beams of the South Broadway Bridge in Kansas, USA. Recently, bonded FRP laminates were used by the New York State Department of Transportation in a demonstration project to repair girders of a concrete T-beam bridge to increase their flexural and shear capacities. Total cost of the rehabilitation was estimated at $300,000 in contrast to $1.2 million required for complete structural replacement (Hag-Elsafi, et al. 2001). The review on FRP applications is summarized in Table I.A.1.4.

Table I.A.1.4 Applications of FRP-Concrete.

No.	Title	Author / Authors	Source	Objectives	Parameters Evaluated	Selected Outcome/Conclusions
1	On the parameters influencing the performance of reinforced concrete beams strengthened with FRP plates	Taheri, F., Shahin, K., and Widiarsa, I.	Composite Structures, v 58, 2002, p 217-226	To assess the influence of various physical and mechanical parameters on the performance of RC beams strengthened with FRP plates	FRP plate length, fiber orientation, surface preparation Poisson ratio mismatch between concrete and FRP plate	Applying FRP plate as long as its hosting RC beam is ideal. Unidirectional and cross-ply GFRP plates provide almost identical improvements in strength. Beams strengthened with long plates and with no surface preparation have lower ultimate loads and slightly lower stiffness than those with surface preparation. The higher the Poisson's ratio mismatch between the FRP plate and the RC beam, the earlier the likelihood of onset of delamination. Stiffer reinforcement is not necessarily better.
2	External reinforcement of concrete beams using fiber reinforced plastics.	Ritchie, P.A., Thomas, D.A., Lu, L.W., and Connelly, G.M.	ACI Structural Journal, v 88, n 4, 1991, p 490-500.	To study the effectiveness of external strengthening using FRP plates	Surface preparation, strength of adhesive, plate length	The surface to be bonded must be clean. Sandblasting for the surfaces of concrete and FRP plates is preferred. The epoxy should have bond strength of at least that of the concrete. Plates must be long and thin to avoid undesirable brittle plate separation failure. FRP plates bonded to the tension zone increase beam stiffness by 17-99% and ultimate strength by 40-97%.
3	Application of FRP laminates for strengthening of a reinforce-concrete T-beam bridge structure	Hag-Elsafi, O., Alampalli, S., and Kunin, J.	Composite Structures, v 52, 2001, p 453-466	Evaluate effectiveness of the strengthening FRP system and investigate its effect on the bridge structural behavior	---	After installation of FRP laminates, main rebar stresses were moderately reduced, concrete stresses (flexural and shear) moderately increased, and transverse live-load distribution to the beams slightly improved under service loads. Location of the neutral axis migrated down after the laminates were installed.

No.	Title	Author / Authors	Source	Objectives	Parameters Evaluated	Selected Outcome/Conclusions
4	Structural behavior of composite RC beams with externally bonded CFRP	Spadea G, Bencardino F, and Swamy RN	Journal of Composite Construction, v 2, n 3, 1998, p 132-137	Behavior of reinforced concrete beams strengthened with bonded CFRP plates	End-anchorage	Up to 70% increase in load capacity was observed when external anchorages were used.
5	Effects of wrap thickness and ply configuration on composite-confined concrete cylinders	Parvin, A., Jamwal, A.	Composite Structures, v 67, 2005, p 437–442	Behavior of small-scale FRP wrapped concrete cylinders under uniaxial compressive loading	FRP wrap thickness, and ply configuration.	The cylinders with ‘‘hoop-angle-hoop’’ ply configuration in general exhibited higher axial stress and strain capacities as compared to the cylinders with the ‘‘angle-hoop-angle’’ ply configuration. The increase in wrap thickness also resulted in enhancement of axial strength and ductility of the concrete cylinders.
6	Retrofitting of shear failed reinforced concrete beams	M.N.S. Hadi	Composite Structures, v 62, 2003, p 1–6	To investigate the effectiveness of two types of wrapping material in enhancing the shear capacity of RC beams, and the increase in the strength and ductility of RC beams	FRP types, and wrapping layers	The more layers of FRP materials applied, the higher flexural strength and shear capacity would be achieved. The beams retrofitted with E-glass achieve a shear capacity enhancement up to 17%. Under the same amount and configuration, the CFRP material outperforms the E-glass material in structural externally strengthening.
7	Shape and ‘‘gap’’ effects on the behavior of variably confined concrete	Kent A. Harries, Shawn A. Carey	Cement and Concrete Research, v 33, 2003, p 881–890	To refine the understanding of factors affecting the behavior of confined concrete	Jacket bond to concrete surface and cross section shape	The provision of a gap results in a reduced maximum attainable concrete strength. The jacket efficiency was not affected by the provision of the gap. Square specimens exhibit lower confinement levels than circular specimens having the same jacket.
8	Strength and strain capacities of concrete compression members reinforced with FRP	G. Campione, N. Miraglia	Cement & Concrete Composites, v 25, 2003, p 31–41	Exam the compressive behavior of concrete members reinforced with FRP	The shape of the transverse cross-section of concrete members	The effectiveness of FRP reinforcement is less with a square section compared to a circular cross section. A significant increase in maximum strength was observed for the circular cross-section; instead less effective confinement was observed for square sections or square sections with round corners.

No.	Title	Author / Authors	Source	Objectives	Parameters Evaluated	Selected Outcome/Conclusions
9	The mechanical behaviour of composite-wrapped concrete cylinders subjected to uniaxial compression load	Lau KT, Zhou LM	Composite structures, v 52, 2001, p 189-198	Present the behavior of the wrapped concrete cylinder with different wrapping materials and bonding dimensions	Wrapping materials and bonding dimensions	The deflection of the wrapped concrete cylinder in the load direction decreases with increasing the length, thickness and modulus of the wrapping sheet. Using a longer wrapping sheet with high modulus may not benefit the structure since it creates negative hoop stress in the wrapping sheet.
10	Structural performances of short steel-fiber reinforced concrete beams with externally bonded FRP sheets	J. Yin, Z.S. Wu	Construction and Building Materials, v 17, 2003, p 463–470	Present an approach to improve the FRP strengthening performance to concrete beams by mixing short steel-fibers into the concrete matrix	---	Mixing short steel fibers into concrete can greatly improve concrete toughness. The crack propagation in concrete can be controlled. Such an improvement can have FRP sheet providing better strengthening effect. It could effectively prevent the rapid propagation of localized concrete cracking. The failure mode may change from interfacial debonding to FRP rupture.
11	The effect of surface preparation on the bond interface between FRP sheets and concrete members	Toutanji, H. A., and Ortiz, G.	Composite Structures, v 53, 2001, p 457-462.	Present the influence of concrete surface treatment and the type of FRP sheets on the bonding strength of concrete-FRP sheet	Type of fiber and surface treatment	Surface treatment by water jet produces a better bonding strength than surface treatment by sander. Specimens reinforced with glass fiber sheet showed a lower average tensile stress than those reinforced with high modulus carbon fiber sheets.
12	Strengthening concrete beams for shear with CFRP sheets	B. Taljsten	Construction and Building Materials, v 17, 2003, p 15-26	Presents examples to strengthen concrete beams for shear	Direction and thickness of fibers or laminates	Concrete beams can be strengthened for shear and fabrics or laminates should be placed perpendicular to the shear crack if possible. The thinner the fiber used, the better the utilization of the fabric. A ratio of approximately 0.55 of the maximum measured strain value can be recommended for engineering design.
13	Canadian Highways Bridge Design Code, Section 16, Fiber Reinforced Structures	---	Canadian Standards Association, 1996.	Design guidelines	Maximum tensile strain	The limit of tensile strain in FRP shear reinforcement is 0.2%.

No.	Title	Author / Authors	Source	Objectives	Parameters Evaluated	Selected Outcome/Conclusions
14	Investigation of bond in concrete structures strengthened with near surface mounted carbon fiber reinforced polymer strips	T. Hassan and S. Rizkalla	Journal of composite for construction, v 7, n 3, 2003, p 248-257	Evaluate bond characteristics of near surface mounted carbon FRP strips	Internal steel reinforcement ratio, concrete compressive strength, groove width	The ultimate load carrying capacity increased up to 53%. Grooves with 5mm wide by 25mm deep were adequate to prevent splitting of epoxy cover. Debonding loads increased by increasing the embedment length of CFRP strips, concrete compressive strength, and/or groove width. Development length of near surface mounted CFRP strips increased by increasing the internal steel reinforcement ratio, and decrease with the increase of either concrete compressive strength and/or groove width.
15	Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures	---	ACI Committee 440. Framington Hills MI: American Concrete Institute, 2002.	Design guidelines	Conservative strength reduction factors, maximum tensile strain	A strength reduction factor of 0.5 for rupture-controlled failures and 0.7 concrete crushing failures is suggested. The maximum tensile strain in FRP shear reinforcement is 0.2%.
16	Recommendation for design and construction of concrete structures using continuous fiber reinforcing materials	---	Japan Society of Civil Engineers (JSCE), Concrete Engineering Series, 1997: 23.	Design guidelines	Conservative strength reduction factors	A strength reduction factor of 1/1.3 for design of flexural members using FRP is suggested.
17	Flexural response of concrete beams reinforced with FRP reinforcing bars	Benmokrane B, Chaallal O & Masmoudi R	Structural Journal, v 93, n 1, 1996, p 46-55.	Design guidelines	Conservative strength reduction factors	A strength reduction factor of 0.75 determined on the basis of probabilistic concepts is suggested.

No.	Title	Author / Authors	Source	Objectives	Parameters Evaluated	Selected Outcome/Conclusions
18	Comparison of experimental shear data with code predictions for FRP prestressed beams	Dowden DM, Dolan CW	Proceedings of the 3rd International Symposium on Non-metollic (FRP) Reinforcement for Concrete Structures (FRPRCS-3), Saporo, Japan, 1997, p 687-694.	Design guidelines	Maximum tensile strain	The limited value of the shear strain in FRP shear reinforcement is 0.25%.
19	Behavior of externally confined high-strength concrete columns under eccentric load	J.Li, M.N.S.Hadi	Composite Structures,2003, 62: 145-153	Study the behavior of externally reinforced high-strength concrete columns subjected to eccentric loading	Effect of FRP layer number, and FRP types on strength of concrete	No. of layers of FRP has a significant effect, carbon fibers had significantly better effect on the normal strength concrete while for high strength concrete columns no significant benefits were achieved
20	Characterization of debonding energy release rate of FRP sheets bonded on mortar and concrete	Kimpara I, Kageyama K, Suzuki T, Osawa I, Yamaguchi K	Advanced Composite Materials, 1999, 8 (2): 177-187	Characterize peel strength and examine the effects of different surface treatments and primer	Surface treatments	The different effects of the surface treatments of concrete on the peeling strength of FRP sheets
21	Composites: A new possibility for the shear strengthening of concrete, masonry and wood	T. C. Triantafillou	Composites Science and Tech., 1998, 58:1285-1295	To evaluate the use of composites as shear strengthening materials for concrete, masonry and wood members	---	A significant improvement in the shear capacity of the beams strengthened with composites
22	Reinforced concrete rectangular beams strengthened with CFRP laminates	Sahawany M, Arockiasamy M, Beitelman T, and Sowrirajan R	Composites, 1996, 27(B): 225-233	To evaluate the effectiveness of external reinforcement in terms of the cracking moment, maximum moment, deflection, and crack patterns	Number of CFRP layers	For one, two, and three layers of CFRP, the cracking moment increased 12%, 61%, and 105%, respectively. Deflection decreased inversely with increasing number of CFRP layers on each beam.

No.	Title	Author / Authors	Source	Objectives	Parameters Evaluated	Selected Outcome/Conclusions
23	External reinforcement of concrete beams using fiber reinforced plastics	P. Ritchie, D. Thomas, W. L. Lu, G. Connelly	ACI Structural Journal, 1991, 81: 490-500	To evaluate the performance of external strengthening using three different types of FRP: glass, carbon, and aramid fibers	FRP types	Increase in stiffness from 17% to 99% and an increase in strength of 40% to 97% based on the type, amount, and orientation of FRP that was applied to the beam
24	Deformation in concrete with external CFRP sheet reinforcement	J.M. Tripi, C.E. Bakis, T.E. Boothby and A. Nanni	Journal of Composites for Construction, 2000, 4(2): 85-94	To evaluate the local and global deformations of tensile loaded and cracked reinforced RC beams externally reinforced with CFRP	Thickness of adhesive layers	Thicker adhesive layers allowed larger relative displacements to occur between the CFRP and concrete and caused a more gradual transfer of loads from CFRP to concrete.
25	Behavior of full-scale reinforced concrete beams retrofitted for shear and flexural with FRP laminate	D. Kachlakev and D.D. McCurry	Composites Part B: engineering, 2000, 31: 445-452	To evaluate the effect of varying configurations of CFRP and GFRP composites on the load and strain strain behavior of retrofitted concrete beams	FRP types	CFRP reinforced concrete beams showed higher load-carrying capacity in comparison to the GFRP reinforced beams
26	FRP composites for shear strengthening of RC beams	A. Khalifa, L. D. Lorenzis and A. Nanni	Proceedings of International Conference on ACMBS, Ottawa, Aug. 15-18, 2000	To evaluate the shear strength and mode of failure of FRP strengthened concrete beams	---	CFRP reinforced concrete beams showed an increase of 44% in the ultimate load over the control specimens
27	A new method for evaluating the surface roughness of concrete cut for repair or strengthening	A.I.Abu-Tair, D.Lavery, A.Nadjai, S.R.Rigden, and T.M.A. Ahmed	Construction and Building materials, 2000, 14: 171-176	To study the effect of surface roughness on the bond between a repair material and the concrete surface	Surface preparation	The rougher surface preparations gave a higher strength than the samples with the smooth surfaces. The needle gunned, hand-cut and as fractured surface samples gave almost identical strengths with less than 6% difference. Their average strength was 29% higher than for the smooth surfaces

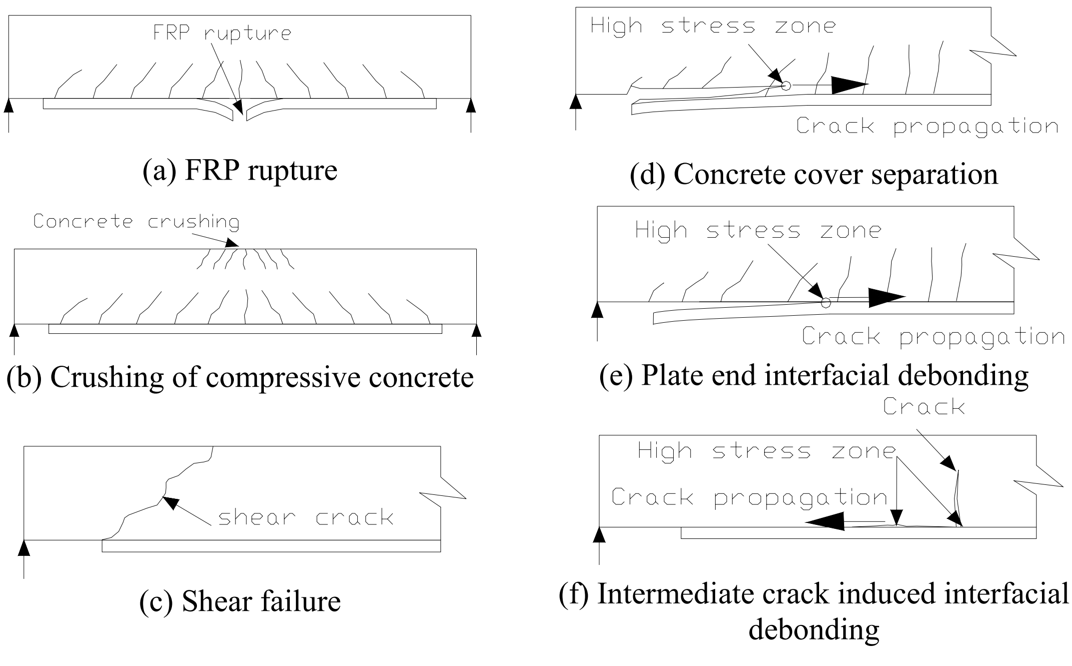
I-A.1.3.2 Structural Response

The structural response of FRP strengthened members are based on limit states design principles, with conservative strength reduction factors for flexure, shear and bond specified in design guidelines (e.g. ACI Committee 440) due to the potential sudden and brittle failure of composites exhibiting linear elastic behavior to ultimate load. Also, in many instances serviceability criteria, and fatigue and creep rupture endurance limits may control the design. The design practice being used for conventional RC beams based on equilibrium principles has been extended to FRP-strengthened beams.

Nanni A. and Norris M.S. (1995) evaluated experimentally the behavior of concrete members laterally confined with FRP composites. Two types of specimen failure were observed. Unconfined specimens experienced failure by propagation of diagonal-shear cracks, jacketed specimens experienced flexural tension and compression failure. Concrete jacketing is effective in enhancing strength and ductility of reinforced concrete members subjected to flexure and combined flexure-compression. The level of strength enhancement is related to the level of axial compression. The tape wrapping method was more effective than the preformed shell method. Circular-section specimens showed larger enhancements than rectangular ones.

Smith and Teng (2002a) observed six main failure modes in tests for RC beam bonded with an FRP soffit plate, as shown in Fig. I.A.1.1. These were termed (a) flexural failure by FRP rupture, (b) flexural failure by crushing of compressive concrete, (c) shear failure, (d) concrete cover separation, (e) plate end interfacial debonding, and (f) intermediate crack induced interfacial debonding.

The first three failure modes are not totally different from those in conventional RC beams. The three failure modes shown on the right are modes unique to beams bonded with a soffit plate. These modes have often been referred to as premature debonding failure modes, as they occur before the flexural failure of the section in mode (a) or (b) or the shear failure in mode (c) occurs. The three modes of debonding shown in Figs. I.A.1.1(d)-(f) can be broadly classified into two types: (a) plate end debonding that initiates at or near one of the plate ends, and (b) intermediate crack induced interfacial debonding that initiates at an intermediate flexural or flexural-shear crack and then propagate from such a crack towards the plate end. The first type of debonding is referred to as *plate end debonding* and the second is referred to as *intermediate crack induced interfacial debonding*. Of these two failure modes, plate end debonding is by far the more commonly reported failure mode. Although less commonly reported, failures by intermediate crack induced debonding are likely to control the strength of a significant portion of FRP-strengthened beams. The design approach is to selectively control more desirable and predictable failure modes ((a)-FRP rupture, (b)-concrete crushing and (c)-shear) while avoiding premature debonding failure modes ((d) and (e)) and flexural induced debonding (f).



(a) FRP rupture

(d) Concrete cover separation

(b) Crushing of compressive concrete

(e) Plate end interfacial debonding

(c) Shear failure

(f) Intermediate crack induced interfacial debonding

Fig. I.A.1.1 Failure modes of FRP-strengthened RC beams (Smith and Teng, 2002a).

End cover separation and shear crack debond are the two most critical debonding modes in beams retrofitted with fiber reinforced polymer composites due to the brittle nature of the failures. However, these failures are still not fully understood. A testing program including 18 rectangular reinforced concrete beams was carried out by Pham and Al-

Mahaidi (2004) to investigate the failure mechanisms and the influence of several parameters on these debond modes. Test results showed that end cover separation starts from FRP ends and fails in the form of shear failure at steel reinforcement level at the root of the concrete teeth between shear cracks. Shear crack debond failure is due to the opening of one of those inclined cracks. Mid-span and end debond are the result of the high shear stress level in concrete (around 1MPa). The performance of FRP was influenced mainly by two important parameters: the ratio of FRP bond length in shear span to concrete depth and the ratio of laminate stiffness to tension reinforcement stiffness. The efficiency of FRP increases with the bond length (or the first ratio) and decreases with the amount of FRP (or the second ratio). Concrete cover and amount of shear reinforcement have insignificant influence on debonding. Steel clamps provide good method to avoid end debond. It does not prevent mid-span debond but it helps the beam ductility by holding the delaminated fabrics to concrete by friction. After debonding of FRP, RC beams still have their original strength as without bonding FRP.

It was found that plate peeling at the plate/glue/concrete interface is, indeed, very rare due to strong chemical bonding of adhesive materials to the concrete (Lau et al. 2001). This type of failure is usually attributed mostly to bad workmanship. However, a high peel-off stress may cause the plate and concrete to separate as a unit from the underside of the steel reinforcement.

Consistent with Fig. I.A.1.1, El-Mihilmy et al. (2000) reported that reinforced concrete beams strengthened with externally bonded FRP reinforcement can exhibit primarily three flexural failure modes (see Fig. I.A.1.2 for a qualitative depiction of these):

(1) Crushing of the concrete in compression before yielding of the reinforcing steel - *compression failure*: $f_s < f_y$;

(2) Yielding of the reinforcing steel in tension followed by rupture of the FRP laminate -*FRP rupture*: $A_f < A_{f,min}$;

(3) Yielding of the reinforcing steel in tension followed by concrete crushing-*tension failure*: $f_s = f_y$.

In addition, several other failure modes are possible:

(4) Shear/tension failure of the concrete substrate-*anchorage failure*;

(5) Diagonal tension failure resulting from shear in the section-*shear failure*;

(6) Debonding of adhesive bond line due to vertical section translations resulting from cracking-*debonding failure*.

To avoid the compression failure mechanism, the cross-sectional area of the bonded FRP laminate cannot exceed the maximum FRP cross-sectional area for the section $A_{f,\max}$, as illustrated by line cd in Fig. I.A.1.2. By assuming that the beam is properly detailed with respect to FRP plate anchorage, shear reinforcement, and epoxy bonding so as to preclude failure modes 4–6, the only possible failure modes are tension failure and FRP rupture. Although the tension failure mechanism is preferable, because it is more ductile, the FRP rupture mode can dominate if the bonded FRP cross-sectional area is less than the minimum FRP area for the cross section, $A_{f,\min}$.

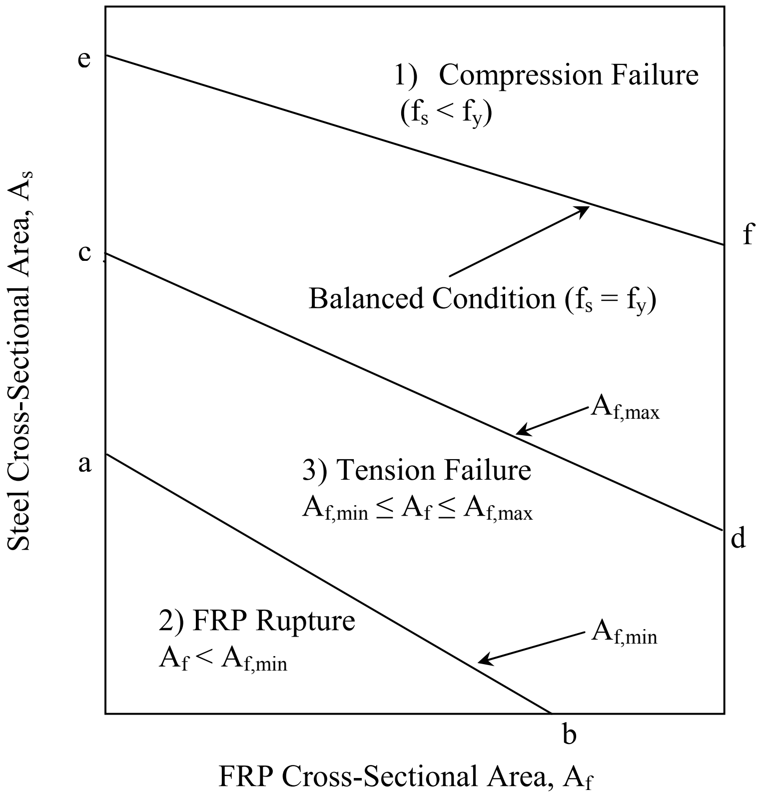


Fig. I.A.1.2 Qualitative-Three Modes of Failure (El-Mihilmy et al. 2000).

Using equilibrium concepts, Professor Davalos and his research team at West Viginia University recently applied a modified design procedure following the guidelines by El-Mihilmy et al. (2000) to predict the ultimate loads for 10 independent experimental studies. The test-beams are simply-supported and subjected to 4-point bending loads.

Nine have rectangular shapes of varying dimensions and one is a T-section. The reinforcement is either glass (GFRP) or carbon (CFRP) as plates or fabrics. The details are given in Table I.A.1.5, and the predicted (P_n) and experimental (P_{exp}) ultimate loads are in good agreement, with an absolute percent mean difference of 8.7%. The FRP debonding, not considered in this analysis, is of great importance and is reviewed in the next section.

Table I.A.1.5 Comparison of Experimental and Predicted Failure Load.

Ref.	$f_c'^{*}$	f_y^{*}	$A_s^{\dagger}$	$d^{\ddagger}$	$b^{\ddagger}$	f_{fu}^{*}	$A_f^{\dagger}$	$d_f^{\ddagger}$	$l^{\ddagger}$	$P_{exp}^{\#}$	$P_n^{\#}$	%
Chajes et al. 1995 (S2)	38.5	413	71	50.8	127	3410	14	76.3	406.4	21.8	21.7	-.31
David et al. 1998 (P6)	40	500	308	268	150	55ж	900	303	900	132	116	-13.77
Ehsani et al. 1990 (large)	36.4	55	260	405	200	380	900	463	2132.5	186	185.4	-.32
Hutchinson et al. 1996 (B:carbon 0.4)	60	575	156	120	200	1532	60	150	750	54	61.2	11.71
Juvandes et al. 1998 (B.1)	36	190	14.1	112	75	2400	60	182	650	31.2	33.6	7.06
Nguyen et al. 2001 (A950)	32.1	384	1161	120	120	3140	96	151	440	56.2	62.5	10.03
Ross et al. 1999 (3)	54.8	414	400	152	200	2206	90	200	914	109	126.4	13.76
Saadatmanesh et al. 1990 (C)	36.4	414	71	130	88.9	400	484	151	685.6	60	70.0	14.33
Saadatmanesh et al. 1991 (T beam)	35	456	1013	400	610	400	912	458	1982.5	310	336.0	7.73
Varastehpour and Hamelin 1995 (B111)	43.5	0	0	70	70	450	52.5	70.4	70	41	44.4	7.74
											Absolute Mean:	**8.68**

*MPa; †mm^2; ‡mm; $^{\#}$KN; ж delamination of plate

A closed-form analytical solution to predict the entire debonding process of FRP-to-concrete bonded joints was presented by Yuan, et al. (2004). The solution provided closed-form expressions for the interfacial shear stress distribution and load–displacement response for different loading stages and provided a rigorous and complete theoretical basis for understanding the full-range load–displacement behavior of FRP-to-concrete bonded joints. The following conclusions were drawn by them: (a) The load–displacement behavior of a bonded joint features a linear elastic stage, a softening stage, a debonding propagation stage, and a linear unloading stage; (b) The ductility of the load–displacement behavior of a bonded joint increases with the bond length but decreases with the plate axial stiffness; (c) The ultimate load of bonded joints increases with the bond length before the effective bond length is reached and remains constant afterwards. It also increases with the plate axial stiffness.

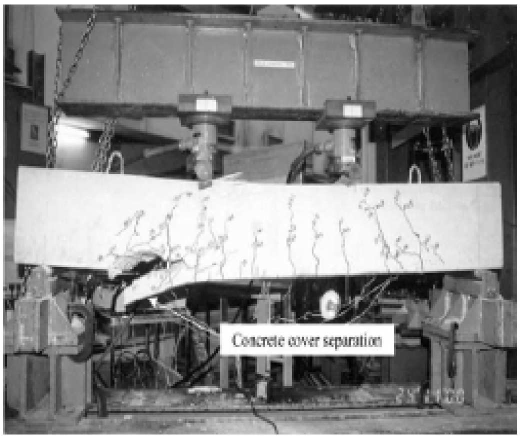
Based on the structural evaluation of full-scale tests conducted on unreinforced and reinforced concrete slab specimens strengthened with carbon/epoxy and E-glassy/epoxy composite systems, the FRP systems have been shown to be successful in upgrading the structural capacity of both two-way unreinforced and reinforced concrete slabs (Mosallam and Mosallam, 2003). For repair applications of unreinforced concrete slabs, test results indicated that the composite system restored not only the original capacity of the damaged slabs but also resulted in an appreciable increase of the strength of the repaired slabs to an average increase of more than 540% the original capacity of the as-built slabs. For retrofitting applications, the use of FRP systems resulted in appreciable upgrade of the structural capacity of the as-built slabs up to 500% for unreinforced specimens and 200% for steel reinforced specimens. In all cases, the failure was preceded by relatively large deformations (more than 1/45 of the clear span length) which provided enough visual warning before ultimate failure. Moreover, no failure occurred in the composite system (with the exception of the debonding of the E-glass/epoxy system for slab E-RETU1); instead a localized compression failure of the concrete was the common failure mode with some localized debonding near the ultimate load, especially for unreinforced concrete slabs retrofitted with E-glassy epoxy system.

I-A.1.3.3 Bond Interface

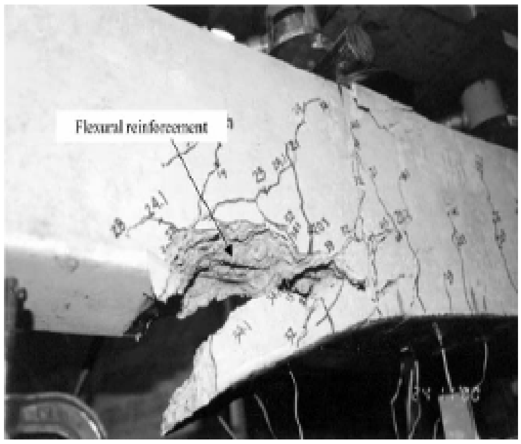
The interface performance is concerned with FRP plate or fabric debonding or separating from the concrete substrate. Debonding failures are very common in FRP-concrete members, and are typically brittle, with no visible warning, and at load levels lower than the flexural or shear strength of the system. At present, however, the design provisions are not well developed, and the approach is mainly to avoid this mode of failure as illustrated above. The research work on debonding has followed three main approaches (Rizkalla et al. 2003): (1) empirical models based on tests results, with inconsistency of results and large scatter; (2) mechanistic or analytical models, but limited to linear elastic behavior; and (3) fracture mechanics approaches, which offer great promise for better understanding of behavior, but are not readily applicable in practice.

There are two major approaches that have been adopted by researchers to predict debonding failure for plated-RC beams: analytical method and numerical method. A complete literature review of these two methods is presented as following.

The ***analytical methods*** have been directed mainly to the problem of *plate-end debonding* (see Fig. I.A.1.1 (d) and (e)), and to a lesser extend to *mid-span debonding* (Fig. I.A.1.1 (f)). Of the two plate end debonding failure modes, failure by concrete cover separation (case (d)) is far more commonly reported (Ritchie et al. 1991, Sharif et al. 1994, Garden et al. 1997, Tumialan et al. 1999, Nguyen et al. 2001). It is generally believed that this failure is initiated by formation of a crack at or near the plate end, due to high interfacial shear and normal stresses caused by abrupt termination of the plate (Varastehpour and Hamelin 1997, Saadatmanesh and Malek 1998, Tumialan et al. 1999, Mukhopadhyaya and Swamy 2001). Once a crack forms in the concrete at or near the plate end, the crack will propagate to the level of the tension reinforcement and then progresses horizontally along the level of the reinforcement, resulting in the separation of the concrete cover. Figure I.A.1.3 shows the detached plate end, where the tension reinforcement of the beam can be clearly seen in the close-up view. Debonding between the FRP plate and the RC beam (Fig. I.A.1.1 (e)), is also believed to be initiated by high interfacial stresses near the plate end that exceed the strength of the weakest element, generally concrete. Upon debonding, a thin layer of concrete often remains attached to the plate (Fig. I.A.1.4). Plate-end debonding failures were first observed in RC beams bonded with steel plates and much research has been done for this case which is also relevant to FRP-plated RC beams.



(a) Overall view



(b) Close-up view

Fig. I.A.1.3 Concrete cover separation of FRP-plated RC beams (Smith and Teng 2002a).

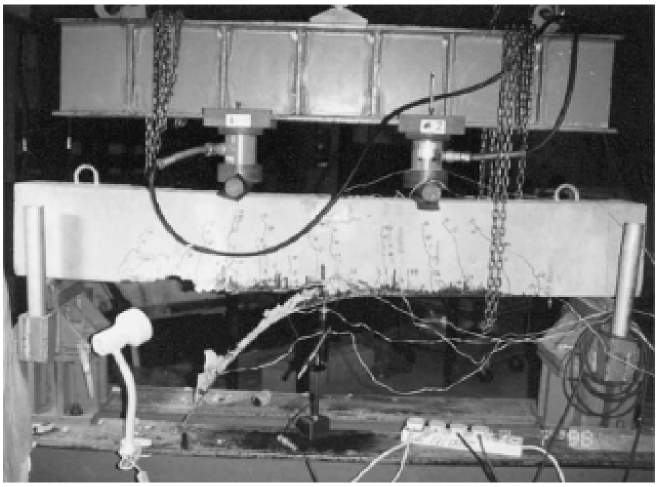


Fig. I.A.1.4 Plate end interfacial debonding of FRP-plated RC beams
(Smith and Teng, 2002b).

Twelve debonding strength models were reviewed by Smith and Teng (2002a), of which seven were developed for FRP-plated RC beams and the other five were for steel-plated RC beams. Since the flexural and shear peeling mechanisms are shown to be the same both for steel-plated and FRP-plated RC beams (Mohamed Ali et al. 2001), earlier developed debonding strength models for steel-plated beams should still be valid for FRP-plated beams after appropriate modifications. These twelve models were classified into three categories based on their approaches, namely (a) shear capacity based models, (b) concrete tooth models, and (c) interfacial stress based models. These models are summarized and reviewed in Table I.A.6. It is worth noting that all the models developed specifically for FRP-strengthened RC beams follow the interfacial stress based approach. Only linear elastic material behaviors are considered, and the key assumption in all of these solutions is that the adhesive layer is subject to shear and normal stresses that are constant across the thickness of the adhesive layer. It is this assumption that enables relatively simple closed-form solutions to be obtained. Based on a relatively large reliable test database consisting of 59 test results for plate-end debonding failures in FRP-strengthened simply-supported RC beams, an assessment of the performance of the above 12 debonding strength models was subsequently presented by Smith and Teng (2002b). They provided comparative evaluations, recommendations with respect to their accuracy, and proposed modifications to achieve better accuracy of predictions as indicated in Table I.A.1.6. They concluded that there is a need for developing a more accurate debonding strength model to reduce the present conservativeness and limitations of existing models.

Although plate-end debonding has been extensively researched, another failure mode, *mid-span debonding* (Fig. I.A.1.1 (f)) has received less attention, but it is equally important. Mid-span debonding initiates at flexural cracks in the central region of the beam and propagates by progressive fracture of a thin concrete layer near the adhesive (about 6-mm thick in Sebastian 2001, Fig. I.A.1.5). The fracture process in concrete is driven mainly by high shear bond stress transmitted to the concrete from the plate by the adhesive. These shear stresses can be generated by any influence that induces axial stress gradients in the plate. There are mainly two sources of axial stress gradients: corrosion of the internal steel reinforcement and tension stiffening in the cracked concrete. Soon after fracture initiation, the mid-span debonding process is self-propagating and can become particularly pronounced after yield of embedded steel reinforcement. In practical applications of strengthening techniques, the use of thin plates with curtailment very near supports, coupled with the presence of large shear span loading, may well encourage mid-span debonding rather than plate-end peel failure in a plated beam. It was pointed out that analysis of the mid-span debonding phenomenon must employ discrete crack modeling and allow for slip of the plate relative to the adjacent concrete, otherwise spurious zero shear bond stresses or unrealistically large shear bond stresses would be predicted.

Table I.A.1.6 Plate End Debonding Models.

No.	Model Name	Author / Authors	Model Category	Theoretical Basis	Debonding Condition	Evaluation/ Recommendation
1	Oehlers's model	Oehlers and Moran 1990; Oehlers 1992	Shear capacity based models	Debonding failure strength is related to the shear strength of the concrete with no or only partial contribution of the steel shear reinforcement. The debonding strength is generally given as the shear force acting at the plate end, with or without taking into account the effect of any coexistent moment.	• The flexural moment at the plate end: $M_{db,f} = (E_c I_{trc,c} f_{ct})/(0.901 E_{frp} t_{frp})$ • The shear force at the plate end: $V_{db,s} = V_c = [1.4 - (d/2000)] b_c d [\rho_s f_c^{'}]^{1/3}$ • Both the shear force and the moment at the plate end: $M_{db,end}/M_{db,f} + V_{db,end}/V_{db,s} \le 1.17$ and $M_{db,end} \le M_{db,f}, V_{db,end} \le V_{db,s}$	For both cover separation and plate end interfacial debonding. Better for concrete cover separation. The most conservative model for pultruded and wet lay-up plate.
2	Jansze's model-for steel plated beams	Jansze 1997	Shear capacity based models	Based on the initiation of shear cracking in an RC beam without the contribution of shear reinforcement.	• The critical shear force in the RC beam at the plate end to cause debonding: $V_{db,end} = \tau_{PES} b_c d$ $\tau_{PES} = 0.18 \sqrt[3]{\frac{3d}{B_{mod}}} (1 + \sqrt{\frac{200}{d}}) \sqrt[3]{100 \rho_s f_c^{'}}$ $B_{mod} = \sqrt[4]{\frac{(1-\sqrt{\rho_s})^2}{\rho_s} d a^3}$	For both cover separation and plate end interfacial debonding. Better for concrete cover separation. Cannot be used for plates terminated at the supports.
3	Ahmed and van Gemert's model-for FRP plated beams (modified Jansze's model)	Ahmed and van Gemert 1999	Shear capacity based models	Based on the initiation of shear cracking in an RC beam without the contribution of shear reinforcement.	• The critical shear force in the RC beam at the plate end to cause debonding: $V_{db,end} = (\tau_{PES} + \Delta\tau_{MOD}) b_c d$ $\Delta\tau_{MOD} = \tau_{PES} b_c d (\frac{S_s}{I_s b_{frp}} - \frac{S_{frp}}{I_{frp} b_a}) + 6188.5(\frac{\tau - 4.121}{b_c d})$ $\tau = (0.15776\sqrt{f_c^{'}} + \frac{17.2366 \rho_s d}{B}) + \frac{0.9 A_{sv} f_{yv}}{s b_c}$	For both cover separation and plate end interfacial debonding. Cannot be used for plates terminated at the supports. Worse to give close and safe prediction.

No.	Model Name	Author / Authors	Model Category	Theoretical Basis	Debonding Condition	Evaluation/ Recommendation
4	Raoof and Zhang's model-for steel plated beams	Zhang et al. 1995; Raoof and Zhang 1997	Concrete tooth models	Use of the concept of a concrete "tooth" between two adjacent cracks deforming like a cantilever under the action of horizontal shear stresses at the base of the beam. Debonding is deemed to occur when these shear stresses lead to tensile stresses at the root of the "tooth" that exceed the tensile strength of the concrete.	• The minimum width of a crack, which leads to a lower bound stress in the plate to cause debond, termed the minimum stabilized crack spacing is given by $l_{\min} = \frac{A_e f_{ct}}{u(\sum O_{bars} + b_{frp})}, u = 0.28\sqrt{f_{cu}}, f_{ct} = 0.36\sqrt{f_{cu}}$ • The effective lengths for end anchorage $L_{p2} = l_{\min}(21 - 0.25 l_{\min}),\ l_{\min} \le 72\text{mm}$ $L_{p2} = 3 l_{\min},\ l_{\min} > 72\text{mm}$	For cover separation only. The most un-conservative model for pultruded and wet lay-up plate. Worse to give close and safe prediction.
5	Wang and Ling's model-for FRP plated beams (modified Raoof and Zhang's model)	Wang and Ling 1998	Concrete tooth models	Use of the concept of a concrete "tooth" between two adjacent cracks deforming like a cantilever under the action of horizontal shear stresses at the base of the beam. Debonding is deemed to occur when these shear stresses lead to tensile stresses at the root of the "tooth" that exceed the tensile strength of the concrete.	• The minimum width of a crack, which leads to a lower bound stress in the plate to cause debond, termed the minimum stabilized crack spacing is given by $l_{\min} = \frac{A_e f_{ct}}{u_s \sum O_{bars} + u_{frp} b_{frp}}$ $u_s = 0.313\sqrt{f_c^{'}},\ f_c^{'} = 0.8 f_{cu}$	For cover separation only. Worse to give close and safe prediction.
6	Raoof and Hassanen's models- for FRP plated beams (modified Raoof and Zhang's model)	Raoof and Hassanen 2000	Concrete tooth models	Use of the concept of a concrete "tooth" between two adjacent cracks deforming like a cantilever under the action of horizontal shear stresses at the base of the beam. Debonding is deemed to occur when these shear stresses lead to tensile stresses at the root of the "tooth" that exceed the tensile strength of the concrete.	The effective lengths for end anchorage: • Raoof and Hassanen's model I $L_{p2} = l_{\min}(24 - 0.5 l_{\min}),\ l_{\min} \le 40\text{mm}$ $L_{p2} = 4 l_{\min},\ l_{\min} > 40\text{mm}$ • Raoof and Hassanen's model II $L_{p2} = l_{\min}(11.6 - 0.17 l_{\min}),\ l_{\min} \le 56.5\text{mm}$ $L_{p2} = 2 l_{\min},\ l_{\min} > 56.5\text{mm}$	For cover separation only. Worse to give close and safe prediction.

No.	Model Name	Author / Authors	Model Category	Theoretical Basis	Debonding Condition	Evaluation/ Recommendation
7	Ziraba et al.'s models-for steel plated beams	Ziraba et al. 1994	Interfacial stress based models	Make use of interfacial stresses from an existing closed-form solution and a concrete failure criterion. The Mohr–Coulomb failure criterion is used to define the critical stress state at plate end interfacial debonding.	• Model I: plate end interfacial debonding. The shear force at the plate end to cause plate end interfacial debonding: $V_{db,end} = \frac{f_c^{'}}{C_{R1}}\left[\frac{C}{\alpha_1 f_{ct}(1+\alpha_2 C_{R2}\tan\Phi)}\right]^{4/5}$ • Model II: concrete cover separation. The shear capacity of an RC beam: $V_{db,end} = (V_c + kV_s)$ $V_c = 1/6(\sqrt{f_c^{'}} + 100\rho_s)b_c d,\ V_s = (A_{sv} f_{yv} d)/s$	Model I: for plate end interfacial debonding only. Worse to give close and safe prediction. Model II: Better for concrete cover separation.
8	Varastehpour and Hamelin's model-for FRP plated beams	Varastehpour and Hamelin 1997	Interfacial stress based models	Make use of interfacial stresses from an existing closed-form solution and a concrete failure criterion. The Mohr–Coulomb failure criterion is used to define the critical stress state at plate end interfacial debonding.	• The shear force in the beam, at the plate end to cause debonding, can be determined by $V_{db,end} = \frac{1.6\tau_{\max}^{2/3}}{\lambda\beta^{1/3}}$, where $\tau_{\max} = \frac{5.4}{1 + C_{R2}\tan 33^0}$ and $\lambda = \frac{t_{frp}E_{frp}}{I_{trc,c}E_c}(d_{frp} - \chi_{trc,c}),\ \beta = \frac{1.26\times10^5 B}{h^{0.7}t_{frp}E_{frp}}$	For plate end interfacial debonding only. Worse to give close and safe prediction.
9	Saadatmanesh and Malek's model	Saadatmanesh and Malek 1998	Interfacial stress based models	Make use of interfacial stresses from an existing closed-form solution and a concrete failure criterion. The Mohr–Coulomb failure criterion is used to define the critical stress state at plate end interfacial debonding.	• Concrete cover separation is deemed to occur when the maximum principal stress (in MPa) reaches the concrete splitting tensile strength, that is when $\sigma_1 = f_{ct} = 0.295\left(f_c^{'}\right)^{2/3}$	For both cover separation and plate end interfacial debonding. Cannot be used for plates terminated at the supports. Worse to give close and safe prediction.

No.	Model Name	Author / Authors	Model Category	Theoretical Basis	Debonding Condition	Evaluation/ Recommendation
10	Tumialan et al's model-similar to the Saadatmanesh and Malek model, but the stresses acting at the plate end are evaluated differently	Tumialan et al. 1999	Interfacial stress based models	Make use of interfacial stresses from an existing closed-form solution and a concrete failure criterion. The Mohr–Coulomb failure criterion is used to define the critical stress state at plate end interfacial debonding.	• Failure by concrete cover separation is deemed to occur when the maximum principal stress reaches the modulus of rupture of the concrete f_r $\sigma_1 = f_r = 0.689\sqrt{f_c'}\,(\text{MPa}) = 8.3\sqrt{f_c'}\,(\text{psi})$	For both cover separation and plate end interfacial debonding. Cannot be used for plates terminated at the supports. Worse to give close and safe prediction.

Nomenclature for Table I.A.6:

E_c : modulus of elasticity of the concrete

E_{frp} : modulus of elasticity of the FRP

t_{frp} : FRP plate thickness

V_c : concrete shear capacity

d : effective depth of the section

b_c : section width

ρ_s : ratio of steel tension reinforcement

f_c' : concrete cylinder compressive strength

B : shear span

b_a : width of adhesive

S_{frp} :first moment of area of the FRP plate

b_{frp} :width of FRP

A_e : area of concrete in tension

$f_{ct} = 0.36\sqrt{f_{cu}}$

f_{cu} : concrete cube compressive strength

α_1, α_2 : empirical multipliers

Φ : angle of internal friction

k : efficiency factor of steel shear reinforcement

S_s : first moment of area of an equivalent steel plate

a :distance from the support to the nearer end of the soffit plate

$\sum O_{bars}$: total perimeter of the tension reinforcement bars

u_{frp} : average bond shear strength between FRP and concrete

C_{R1}, C_{R2} : obtained from Rober's analytical solution

C : constant based on experimental and numerical findings

$\chi_{trc,c}$: neutral axis depth of a cracked section transformed to concrete

h : depth of beam

f_{ct} : cylinder splitting tensile strength of concrete

$I_{trc,c}$: cracked second moment of area of the plate section transferred to concrete

d_{frp} : distance from the compression face of the RC beam to the centroid of the FRP plate

I_{frp}, I_s : second moment of area of a cracked plated section transferred to concrete with an FRP plate and an equivalent steel plate respectively

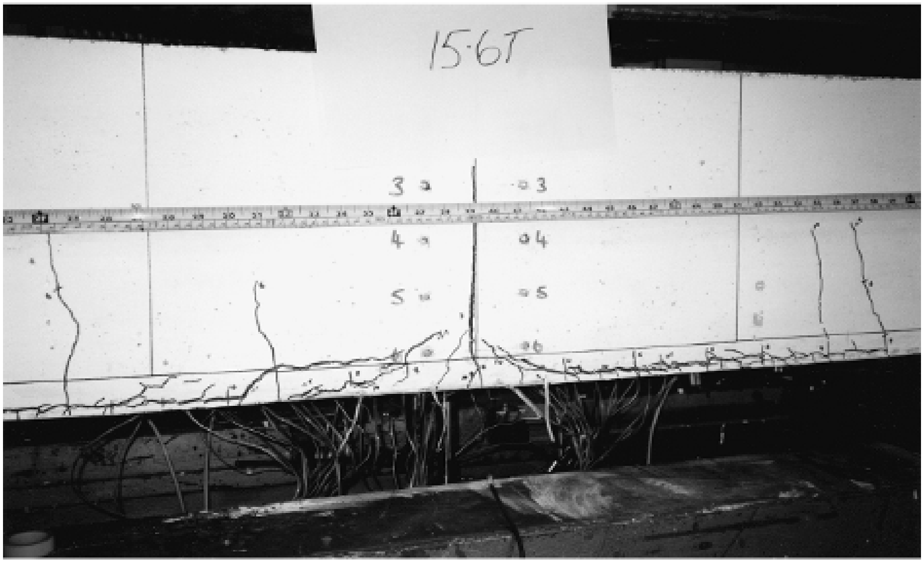


Fig. I.A.1.5 Mid-span debonding of FRP-plated RC beam (Sebastian 2001)

Numerical methods can overcome the simplifying assumption of uniform stresses across the thickness of the adhesive layer as adopted in the above analytical methods. In particular, Finite Element analysis can provide an accurate description of stress state in the adhesive layer.

Teng et al. (2002) presented a detailed finite element investigation into interfacial stresses in reinforced concrete RC beams strengthened with a bonded soffit plate. The results enabled a comprehensive assessment of the performance of the approximate closed-form analytical solution of Smith and Teng (2001) which is representative of a number of similar solutions (Liu, Zhu 1994, Taljsten 1997, Malek et al. 1998), and identified a number of aspects not reflected by such analytical solutions. The following are the main conclusions. (1) Points of stress singularity exist in a plated RC beam. (2) Stresses varied strongly in the adhesive layer, with stresses along the adhesive-concrete (AC) interface being very different from those along plate-adhesive (PA) interface. Specifically, near the end of the plate, the interfacial normal stress was tensile along the AC interface but compressive along the PA interface, which can explain the fact that PA interfacial failure in tests has rarely been reported. (3) Approximate analytical closed-form solutions, being based on the assumptions of uniform stress distributions across the adhesive layer thickness, can provide average predictions for the interfacial stresses. (4) The interfacial stresses increased with a reduction in adhesive thickness, and an increase in adhesive elastic modulus, plate thickness or elastic modulus. The locations of the peak normal and shear stresses were found to depend strongly only on the adhesive layer thickness. The same trend has been noted by Rabinovich and Frostig (2000). (5) The effect of a spew fillet at the end of the plate formed from excess adhesive can reduce the interfacial stresses. The concave spew fillet is shown to be the most effective fillet in terms of reducing the interfacial stresses at the end of the plate followed by the linear spew fillet and then the convex fillet.

Arduini and Nanni (1997) developed a discrete element method that accounts for the mechanical properties of the constituent materials and the characteristics of the concrete-to-FRP interface. The resultant tensile forces are found from a sectional moment – curvature analysis at the ends of each discrete element. This approach can be used to find the maximum bond stresses at the FRP plate and concrete interface as well as the bond stress distribution at the interface throughout the beam length. This approach is computationally attractive but it has the disadvantage that it does not consider the effects of diagonal tension cracking in the shear span of a beam. As a result, the model cannot fully represent the FRP plate–concrete interface bond stress distribution. Based on the work developed by Arduini and Nanni (1997) and incorporating the effects of diagonal tension cracking resulting from the presence of shear forces in a beam, Wang and Chen (2003) analysed the beams externally bonded with FRP laminates to their soffit and sides. The model accurately predicted the load–displacement behavior of the units tested. Excellent agreement was also found between the measured plate strains and the strains predicted by the model.

Two-dimensional interface element method was used by Ziraba and Baluch (1995) and Adhikari and Mutsuyoshi (2002) to study the flexure-shear response of RC beams strengthened externally by epoxy bonded steel plates. Ziraba and Baluch (1995) presented a computational model for analysis by using finite element method (FEM), which consists of a thin layer interface element for epoxy joint between the RC beam and strengthening steel plate. However, their model lacks the effects of end anchors, which are generally used at the plate cut-off zone in practical situations that have significant positive effect against debonding of plate from concrete. Furthermore, the simulations were limited to a few cases and the main parameter dealt with only the plate thickness whereas an important parameter such as plate curtailment distance from the support was not covered. Adhikari and Mutsuyoshi 2002) developed a generalized FEM model for the analysis of RC beams strengthened against flexure by epoxy bonded steel plates at their tensile side. The model takes into account the effect of slip between concrete and the steel plate as well as the effects of end anchors (in the form of anchor bolts) and the non-linear behavior of concrete, reinforcing bars and steel plate. It was confirmed that the new FEM model developed predicts the failure modes of steel plate strengthened RC beams accurately, whereas the assumption of a perfect bond between the concrete and steel plate leads to erroneous results. The load deflection curves and plate strain curves from the analysis with new model show good agreement with the experimental results, whereas the curves from the perfect bond analysis show bad correlation. The assumption of perfect bond between the concrete and steel plate cannot be used in general case of steel-plate-strengthened beams.

Aprile et al. (2001) used a displacement-based fiber beam model to predict the stiffness, load capacity, and failure modes of RC members strengthened in bending with bonded steel or carbon-fiber–reinforced plastic thin plates. Bond slip between the beam and the plate was included by assuming separate displacement fields in the beam and in the strengthening plate. This model is limited to shallow beams, where shear deformations are neglected. They suggested that future studies should concentrate on deep beams, where the interaction between shear stresses and bond slip may cause different failure modes.

In finite element analysis, various procedures have been adopted for predicting cracking in concrete and these fall broadly into two main methods, namely, the disrete crack approach and smeared crack formulation (Rahimi and Hutchinson, 2001). With the discrete crack concept, the position and direction of crack growth within the model is predefined. The smeared crack strategy, however, tends to spread crack formation over the entire structure so that it is incapable of predicting local fracture. Recently, damage models have become popular techniques for simulating various nonlinearity effects in materials. Rahimi and Hutchinson (2001) used an isotropic damage model to simulate the nonlinear behavior of concrete by means of a scalar variable called the damage or degradation parameter. These isotropic damage models do not incorporate the shear retention factor used in the smeared crack models, and they allow for degradation in both tension and compression. The damage threshold is analogous to the yield point in an elastoplastic analysis.

Yang et al. (2003) investigated the behavior of an FRP plated RC beam using a discrete crack propagation model based FEA. The discrete crack approach was used in their study because of the incapability of the smeared approach in simulating premature debonding failure modes in FRP strengthened RC structures. A mixed method combining the G-scaling and displacement control procedures was developed and used in the analyses. The G-scaling procedure was used at first to only allow one crack to initiate at each step. After several loading steps, multiple cracks may initiate. When one of these cracks starts to propagate, the displacement control procedure is used thereafter to allow multiple cracks to propagate within a single step. This mixed algorithm has been successful in their study.

An interface Finite Element based on the Cohesive Zone Model (CZM) has recently been developed at WVU (Wang and Davalos 2003). This element has been successfully incorporated into the commercial FE program ABAQUS (2003), and therefore it is compatible with all the available concrete models (smeared crack model and damage plasticity model) and nonlinear solution techniques. The CZM does not required defining a pre-crack or singularity as conventional FE models, it is economical and efficient because there is no need to refine the FE mesh as the crack front advances, and it combines strength and energy parameters to accurately model interface debonding. The element has zero thickness and is embedded at the interface between FRP-concrete, and the fracture propagation is characterized as a gradual phenomenon in which separation takes place across a cohesive zone resisted by cohesive tractions. Using this element, they predicted the experimental response of concrete beams tested under 4-point bending by Zarnic et al. (1999), that included control RC beams and CFRP-reinforced and steel-reinforced beams. The results for load-deflection are shown in Fig. I.A.1.6. This element is also able to represent the evolution of crack patterns for all cases.

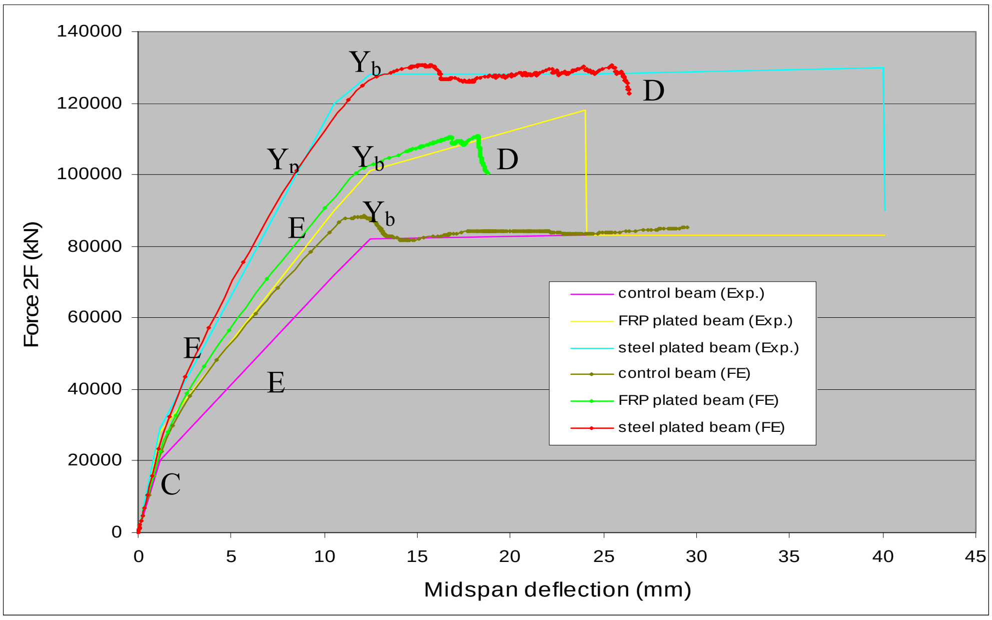


Fig.I.A.1.6 Numerical results with CZM (Wang and Davalos 2003).

I-A.1.3.4 Durability

The durability of the FRP-concrete interface bond is perhaps the major issue of pressing concern, because the long-term adequate strength and integrity of the bond is crucial for the effectiveness of fiber-wrap technology. The literature on durability of interface bonds is inconsistent because of the absence of standardized test methods and protocols, particularly for accelerated weathering conditions. Thus, the reported results are for a variety of conditions and materials, and while most results have shown decrease in bond performance due to weathering exposures, in some instances increases in strength after aging have been surprisingly reported, probably due to lack of proper experimental designs and fundamental understanding of the problem.

Chajes et al. (1995) studied the durability of concrete externally reinforced with three different types of composite fabrics (aramid, E-glass and graphite) subjected to freeze-thaw and wet/dry cycling in a calcium chloride solution. Of the three fabrics, results indicated about 50% reduction in the strength of the aramid and E-glass reinforced beams, while only 5% reduction was observed in the ultimate load of the graphite reinforced beams. Of the two aging protocols, the wet/dry environment was found to cause slightly greater degradation.

Karbhari et al. (1996) evaluated the effect of five different environmental conditions on the durability of FRP plated concrete beams for two different resin and fiber systems; their results indicated that the degradation in performance levels were greater for glass fiber than for carbon, and immersion in salt water caused a greater degradation in stiffness in relation to fresh water.

Green et al. (1998) studied the effect of freeze-thaw cycling on durability of the bond between FRP and concrete, based on experimental results using both single-lap pull-off specimens and beam specimens; they observed that the bond strength was not affected significantly, and surprisingly, the average bond stress value was found to be higher for specimens exposed to increased number of freeze-thaw cycles.

Mukhopadhyaya et al. (1998) evaluated the influence of aggressive exposure conditions on the behavior of epoxy adhesive bonded concrete-GFRP joints. The specimens were subjected to three accelerated aging regimes: (1) alternated wet/dry cycling in 5% sodium chloride solution, (2) freeze-thaw cycling in air for a temperature cycle of 20^{o} C and -17.8^{o} C, and (3) a combination of chloride immersion and freeze-thaw cycling; the combined chloride immersion and freeze-thaw exposure regime was found to induce the largest FRP-concrete differential movement and was considered to be more critical and harmful to the integrity of the adhesive bonded joints.

Toutanji and Gómez (1997) evaluated concrete beams strengthened with four different types of FRP sheets: two types of each carbon and glass FRP reinforcements. Three different types of two-part epoxies were used for the matrix. The specimens were conditioned in two different environments: room temperature (20^o C) and for 300 wet/dry cycles. Debonding of the FRP was a prevalent mode of failure, with different percent reductions in strength. Later, Toutanji and El-Korchi (1998) evaluated performance (under tension) of cement and concrete materials wrapped with carbon and glass FRP sheets subjected to wet-dry and freeze-thaw cycles, showing a 20% decrease in strength for the glass-wrapped specimens under wet/dry exposure. Subsequently, Toutanji and Ortiz (2001) evaluated the influence of concrete surface treatment and the type of FRP sheets on the bond behavior of FRP and concrete. Experimental results indicated that surface treatment by water jet produced a better bonding strength than surface treatment by sander. Specimens reinforced with glass fiber sheet had lower average tensile stress than those reinforced with high modulus carbon fiber, thus indicating better performance of structures externally reinforced with carbon fibers.

Lau et al. (2001) evaluated the structural behavior of GFRP strengthened concrete structures subjected to axial compression and three point bending tests. Pre-cured plate bonding and direct hand lay-up techniques were used to strengthen the concrete beams, and they also evaluated the performance of the specimens when exposed to fresh water, saline water, alkaline, and acidic solutions; the hand lay-up technique was found to give better strengthening characteristics in comparison to the pre-cured plate bonding method. Finally, Malvar et al. (2003) studied the short-term effects of temperature, moisture and chloride content on the CFRP adhesion based on the results obtained from pull-off tests. The objective of their study was to evaluate the effect of high temperature and high humidity on the bond strength. Tests on dollies bonded to aluminum plates (used to test the durability of the epoxy) indicated that tropical exposure (35^o C and 95% RH) could reduce the bond strength to just above the minimum ACI 440 proposed requirement of 1.38 MPa (200 psi). Most of the failures occurred at 38^o C and 95 % RH. It was found that the application of primer generally enhanced the bond strength.

Extensive work on interface bond strength and durability has been conducted by Davalos and coworkers at West Virginia University (Davalos, et al. 1998, 2000, 2002, 2004; Jia and Davalos, 2004; Boyajian et al., 2000, 2002), resulting in the development of conventional and fracture mechanics methods for interface bond characterization of FRP-Wood and FRP-concrete materials.

For FRP-wood, Davalos and WVU co-workers have proposed ASTM modified tests for delamination under wetting/drying cycles, and shear strength evaluations by compression loading, followed by fracture mechanics methods with Contoured Double Cantilever Beam (CDCB) tests for dry and environmentally conditioned samples (Davalos, et al., 1998, 2000, 2002, 2004), including more recently fatigue fracture studies for varying load-ratio, frequency and wave-form effects (Jia and Davalos, 2004). For FRP-concrete, the WVU team has proposed modified approaches for accelerating aging protocols and fracture mechanics methods with a newly developed Single Contoured-Cantilever Beam (SCCB) specimen (see Fig. I.A.1.7, Boyajian et al., 2000, 2002). The weathering regimes they studied included: wet-dry cycles under sodium sulfate (attack from ground contact leading to expansive gypsum and ettringite) and sodium hydroxide (alkalinity attack from hydration of cement); and freeze-thaw under calcium chloride (effects of deicing salts in winter). The novel SCCB specimen was used to obtain interface fracture energy release rate and its degradation under simulated aging conditions. Figure I.A.18 shows that the decrease in interface fracture energy can be as much as about 70% after 30 wet-dry cycles for normal (NC) and high-performance (HPC) concretes. The WVU team has provided guidelines that can be effectively used for product qualification, testing protocol designs, and performance evaluations, and these results can lead to service-life predictions.

A list of compiled references on FRP-concrete durability is given in Table I.A.1.7.

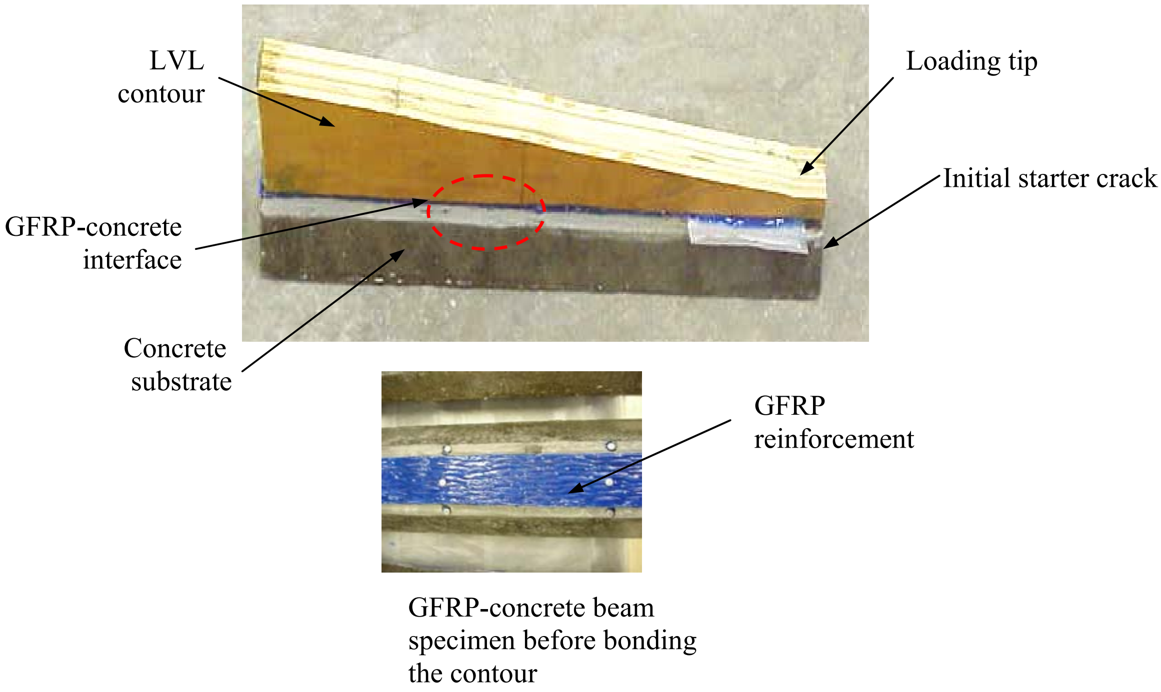


Fig.I.A.1.7 Single contoured-cantilever beam(SCCB) specimen.

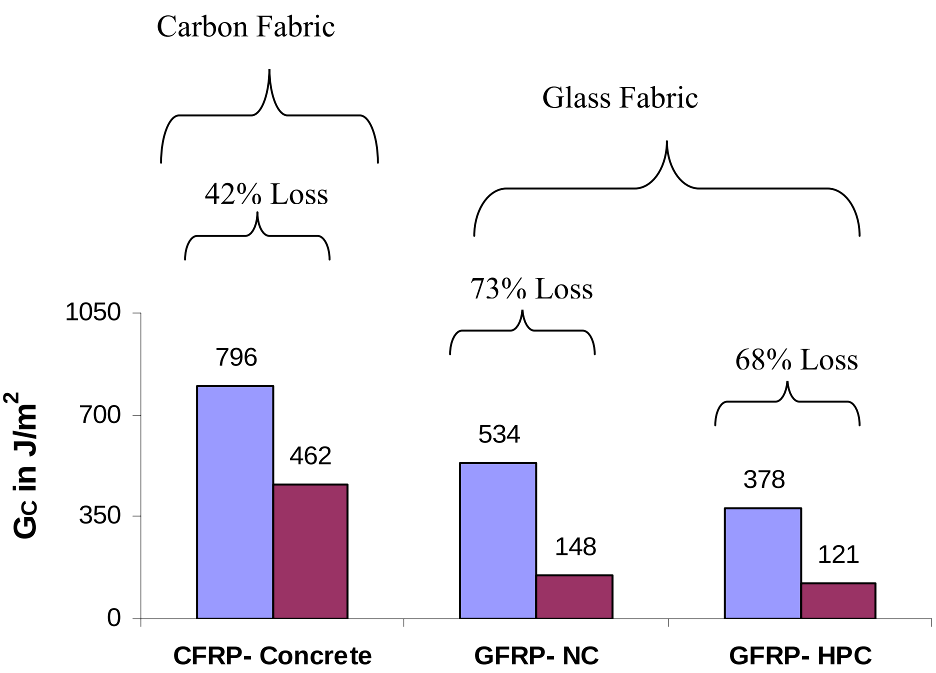


Fig. I.A.1.8 Decrease in Fracture Energy (G_c) after 30 Wet-Dry Cycles.

Table I.A.1.7 A Compiled List of Additional Information on FRP-Concrete Interface Bond Durability.

No.	Title	Author / Authors	Source	Objective	Outcome
1	Characterization of debonding energy release rate of FRP sheets bonded on mortar and concrete	I. Kimpara, K. Kageyama, T. Suzuki, I. Osawa, and K. Yamagudhi	Advanced Composite Materials, 1999, 8(2): 177-187	A new peeling test to characterize the peeling strength of FRP sheets bonded to mortar and concrete	Energy release rate of CFRP-concrete bond was in the range of 590 J/m^2 and 880 J/m^2 depending on the type of surface treatment
2	Performance Evaluation of Reinforced Concrete Bridge Columns Wrapped with Fiber Reinforced Polymers	Teng MH, Sotelino ED and Chen WF	Journal of Composites for Construction, 2003, 7: 23, 83-92	Performance of new bridge columns wrapped with FRP exposed to both aggressive environmental conditions and thermal effects	Among CFRP,GFRP & AFRP, CFRP performed best in terms of structural behavior
3	On the durability of composite rehabilitation schemes for concrete: use of a peel test	V.M. Karbhari, M. Engineer, and D. A. Eckel II	Journal of Materials Science, 1997, 32: 208-227	Develop a peel test for investigation of bond between composites and the concrete surface	The interfacial energy for glass as ranging from 413.6 to 417.92 J/m^2 and that for carbon from 598.68 to 646.61 J/m^2
4	Exploratory evaluation of Mode-I fracture toughness of concrete-composite bonded interfaces	Xu Y, Qiao P and Davalos J	15^{th} ASCE Engineering Mechanics Conference, New York, 6/2/02-6/5/02	Characterize Mode-I fracture of concrete-FRP bonded interface based on the notched three-point bending beam	Critical load and critical fracture toughness of GFRP-concrete is higher than that of CFRP concrete
5	The CFRP-Concrete Interface Subjected to Sodium-Sulfate and-Hydroxide Attack	Boyajian D, Davalos J, Ray I & Kodkani S	17^{th} ASC Technical Conference West, Lafayette, IN, Oct. 21-23, 2002, pp 9	Interface bond reliability when subjected to wetting and drying under two media cases:Na2SO4 & NaOH	At the end of 30 cycles G_Ic values reduced by 61% & 57% for Na2So4 & NaOH respectively
6	Evaluation of interface fracture of concrete externally reinforced with FRP	D.M. Boyajian, J.F. Davalos, I. Ray and P. Qiao	Proceedings of ACMBS, Ottawa, Canada, Aug., 2000, 15-18	Evaluation of interface fracture of concrete externally reinforced with FRP	SCCB method was developed and successfully applied the fracture toughness values for CFRP concrete interface

No.	Title	Author / Authors	Source	Objective	Outcome
7	Effect of freeze-thaw cycles on the bond durability between fibre reinforced polymer plate reinforcement and concrete	M. F. Green, L. A.Bisby, Y. Beaudoin and P. Lanossiére	Canadian Journal of Civil Engineering, 2000, 27: 949-959	Effect of freeze-thaw cycling on the anchorage zone of carbon fibre reinforced polymer plates bonded to reinforced concrete beams	Obtained an increase in ultimate loads and hence an increase in the bond stress with increase in the number of cycles
8	Durability characteristics of concrete beams externally bonded with FRP composite sheets	H. A.Toutanji and W. Gómez	Cement and Concrete Composites, 1997, 19: 351-358	Durability of concrete beams externally reinforced with FRP composite sheets subjected to 300 wet/dry cycles	Specimens subjected to wet/dry conditions showed less improvement than those kept at room temperature.
9	Influence of aggressive exposure on the behavior of adhesive bonded concrete-GFRP joints	P.Mukhopadhyaya, R.N.Swamy, C.J.Lynsdale	Construction and Building Materials,1998, 12: 427-446	Effect of aggressive exposure on the behavior of adhesive bonded concrete-GFRP joint	Exposed specimens showed higher strengths and greater differential movements in the strains.
10	Effect of freeze-thaw cycles on the bond of FRP sheets to concrete	M. F. Green, L. A.Bisby, Y. Beaudoin and P. Lanossiére	CDCC , Montreal, Québec, Canada, 1998, pp 179-190	Effect of freeze-thaw cycling on the anchorage zone of carbon fibre reinforced polymer plates bonded to RC beams	Obtained an increase in ultimate loads and hence an increase in the bond stress with increase in the number of cycles
11	Strength,Durability and health monitoring of composite overlays on civil engineering structures	V. Giurgiutiu, J. Lyons, M. Petrou, S. Dutta & C. A. Rogers	International Composites Expo ICE-98, Nashville,TN, Jan 19-21,1998	To develop a theoretical and experimental program to investigate bond strength, durability and in-service health monitoring	Bond strength, adhesion fracture, crack propagation and durability were being considered and methods for in-situ health monitoring were presented.
12	Experimental fracture mechanics for the bond between composite overlays and concrete substrate	V. Giurgiutiu, J. Lyons, M. Petrou, D. Loub and S. Whitley	International Composites Expo (ICE), Cincinnati, OH, May 10-13,1999	To assess the bond strength of composite overlays to concrete utilizing a fracture toughness test	Specimens with high fracture toughness had a large proportion of crack propagation inside the concrete substrate, while the specimens with lower fracture toughness had crack propagation predominantly at the interface, G_{IC} was 0.252 J/mm

No.	Title	Author / Authors	Source	Objective	Outcome
13	Ultrasonic non-destructive assessment of bonding defects in composite structural strengthenings	F. Bastianini, A. Di Tommaso and G.Pascale	Composite Structures, 2001, 53: 463-467	To develop a non-destructive investigation technique for bonding evaluation in external FRP strengthenings	The proposed ultrasonic technique is mainly based on the evaluation of the amplitude of the defect echo
14	Critical strain energy release rate of interface debonding between carbon fiber sheet and mortar	K. Fukuzawa, T. Numao, Z .Wu, H. Yoshizawa and M. Mitsui	Non-Metallic Rft. for concrete structures Proc.of 3rd intl. Symposium, Sapporo, Japan Vol. 1, Oct. 97	To measure the critical mode II strain energy release for CFRP concrete interface	Critical strain energy release rate is found to be independent of the specimen size within the range of the designed specimens G_{IIC} ranged from 5.01 J/m^2 to 9.12 J/m^2
15	Effect of test methods and quality of concrete on bond strength of CFRP sheet	T. Horiguchi and N. Saeki	Non-Metallic Rft. for concrete structures, Proc. of 3rd intl. Symposium, Sapporo, Japan, Vol. 1, Oct. 97	To evaluate the effect of three different types of bond tests with three different qualities of concrete	High correlation between the bond strength and the compressive strength was confirmed. The bond strength decreases with the decrease in the compressive strength, this decrease is significant in the tensile test and the bending test
16	Ductility considerations in using GFRP sheets to strengthen and upgrade structures	N. Swamy, P. Mukhopadhyaya and C. Lynsdale	Non-Metallic Rft. for concrete structures Proc.of 3rd intl. Symposium, Sapporo, Japan Vol.1 Oct. 97	To evaluate the structural ductility of GFRP-concrete beams based on deflection, curvature and energy absorption	Both deflection and energy based on tension steel yielding can be used as a criterion of ductility to evaluate comparative structural performance of plate bonded RC beams
17	The effect of surface preparation on the bond interface between FRP sheets and concrete members	H. A. Toutanji & G. Ortiz	Composite Structures ,2001, 53: 457-462	To evaluate the influence of concrete surface treatment and the type of FRP sheets on the bonding strength of concrete-FRP sheet	Surface treatment by water jet produces a better bonding strength than surface treatment by sander, specimens reinforced with GFRP showed a lower average tensile stress than those reinforced with CFRP
18	Durability of concrete beams externally reinforced with composite fabric	M. Chajes, T. A. Thomas, Jr., and C. A. Fraschman	Construction and building materials, 1995, 9 (3): 141-145	To determine the durability of the concrete-epoxy fabric system subjected to freeze-thaw and wet dry in calcium chloride solution	Wet/dry was found to be more degradative, reduction in ultimate strength was obtained after environmental cycling, E-glass and aramid were less durable in comparison to the carbon fabric

No.	Title	Author / Authors	Source	Objective	Outcome
19	Mechanical performance of composite-strengthened concrete structures	K. Lau, L. Zhou	Composites : Part B, 2001, 32: 21-31	To study the structural behavior of the glass-fibre composite strengthened concrete structures subjected to uniaxial compression and three point bending tests	The strength of the concrete is highly influenced by the acidic environment, the use of glass-fibre composite for concrete strengthening can improve the flexural strength properties of the degraded RC structures in harsh environment
20	Experimental investigation of bond between FRP and concrete	B. Wan, M. Petrou, and K. Harries	Proc. of Int. Conf. on Composites in Infrastructure, San Francisco , June 10-12, 2002	To evaluate the bond toughness between E-glass FRP overlay and concrete substrate	Measured value for mixed mode COD indicate that the flaw is growing in predominantly mode I conditions with a small component of mode II COD remaining constant throughout the growth process
21	Tensile durability performance of cemetitious composites externally wrapped with FRP sheets	H.Toutanji and T.El-Korchi	2nd International conference on composites in infrastructure, Tuscon AZ, 1998a	To study the tensile performance of cement and concrete materials wrapped with FRPC sheets subjected to wet-dry and freeze thaw cycles	Carbon is superior to glass when exposed to harsh environmental weathering, specimens wrapped with glass fibers exhibited a reduction of 20% and 10% when subjected to wet-dry and freeze-thaw cycling respectively
22	Durability of carbon fiber reinforced polymer/epoxy/concrete bond in marine environment	R. Sen, M. Shahawy, G. Mullins and J. Spain	ACI Structural Journal 1999, 95 (6): 906-914	To evaluate the durability of the epoxy bond formed with concrete and CFRP in marine environment	Bond degradation was least for outdoor exposure and greatest under wet/dry cycles. Surface preparation and proper application of epoxy is essential for the long term integrity of CFRP concrete bond
23	Durability characteristics of concrete beams externally bonded with FRP composite sheets	H. Toutanji and T. EL-Korchi	2^{nd} international conference on composites in infrastructure, Tuscon, AZ, 1998b	To evaluate the performance of concrete beams wrapped with GFRP sheets subjected to wet-dry and freeze-thaw cycles	The glass wrapped specimens subjected to wet/dry aging protocol exhibited a 20% reduction in strength whereas those exposed to freeze-thaw cycling suffered a reduction of 5%
24	Effect of Environmental Exposure on the External Strengthening of Concrete with Composites- Short Term Bond Durability	V. M. Karbhari, and M. Engineer	Journal of Reinforced Plastics and Composites, 1996, 15: 1195- 1216	To evaluate the effect of five different environmental conditions on the durability of FRP plated concrete beams	Degradation in performance levels were greater for the glass fiber reinforced system than for the carbon reinforced system; salt water caused a greater degradation in stiffness than immersion in fresh water
25	Environmental effects on the short-term bond of carbon fiber-reinforced (CFRP) composites	L.J. Malvar, N.R. Joshi, J.A. Beran and T. Novinson	Journal of Composites for Construction, 2003,7(1): 58-63	To evaluate the short-term effects of temperature, moisture and chloride content on the CFRP adhesion to concrete	Bond strength decreased significantly at high temperature and humidity. Most of the failures occurred at 38C and 95 % RH

No.	Title	Author / Authors	Source	Objective	Outcome
26	Freeze-thaw durability of composite materials	Gomez J, Casto B	In: Ehsani MR (ed). Fiber composites in infrastructure: Proceedings of the First International Conference in Infrastructures, Tucson, AZ, 15-17 January 1996.	Durability of CFRP externally attached to concrete beams exposure to cycles of freezing and thawing and wetting and drying cycles	Water enters the cracks and expanded with subsequent freezing, causing bond failure and peeling of the composite from the substrate. When compared to the control beams kept at room temperature, the thermal cycles had no negative effect on the ultimate loads applied to the beam
27	A review of low temperature response of reinforced concrete beams strengthened with FRP sheets	Baumert ME, Green MF & Erki MA	In: El-Badry MM (ed). Advanced composite materials in bridges and structures: 2nd International Conference, Montreal, Quebec, 11-14 August 1996. Montreal: Canadian Society for Civil Engineering. 1996: 565-572	Low-temperature behavior of FRP externally reinforced concrete	The adhesive bond would be damaged and the laminate would peel after being subjected to -60^{o}C; the laminate remained intact. At -27^{o}C, the CFRP sheets increased the flexural capacity of the beam up to shear peeling failure of the sheets indicating a premature failure of the bond.
28	External support of concrete structures using composite materials	Bavarian B, Shively J, Ehrgott R & Di Julio R.	In: Ehsani MR (ed) Fiber composites in infrastrures: Proceedings of the 1st International Conference in Infrastructures, Tucson, Arizona, 15d17 January 1996. Tucson, AZ: University of Arizona. 1996. 917-928.	The effects of elevated temperatures and salt spray on the S-glass embedded in polyester resin and Kevlar-29 embedded in epoxy resin.	Standard concrete cylinders exposed to elevated temperatures (and salt fog application) of 49^{o}C for a period of 28 days with no significant loss of strengthening capability. Compression tests on the conditioned (salt spray) cylinders did not show damage or loss of strength.
29	Performance of CFRP retrofitted concrete columns at low temperatures	Soudki KA & Green MF.	In: El-Badry MM (ed) Advanced composite materials in bridges and structures: 2nd International Conference, Montreal Quebec, 11-14 August 1996. Montreal: Canadian Society for Civil Engineering. 1996: 427-434.	Water and freeze-thaw effects on CFRP wrapped concrete columns	Carbon wraps provided more strength, stiffness, and ductility than for control columns subjected to the same environment. At room temperature, CFRP sheets increased strength by 20% for one layer and 30% for two layers. With thermal cycling, one layer of CFRP tripled the strength of the unwrapped column exposed to the same environment, while a second layer caused the column to fail at the same load as an unwrapped column kept at room temperature. The axial strength of an unwrapped column kept at room temperature was 65% more than an unwrapped column put through thermal cycling.

No.	Title	Author / Authors	Source	Objective	Outcome
30	Durability of FRP reinforcements for concrete	Hamilton III HR, Dolan CW	Prog. Struct. Engng Mater, 2000; 2: 139-145	Ozone effect on durability of FRP wrapped system	There was no apparent effect on the panel strength from the ozone exposure.
31	NOL-ring based evaluation of freeze and freeze-thaw exposure effects on FRP composite column wrap systems	Zhang S, Karbhari VM, Reynaud D	Composites: Part B, 32(2001) 589-598	Evaluate durability of FRP wrap column systems after exposure to freeze (-15^{o}F) and freeze-thaw conditions	Freeze-thaw exposure after salt soak results in reduction in both tensile strength and interlaminar shear strength. The adhesive layer is the weak link. In comparison to freeze-thaw conditions, sustained exposure to -15^{o}F conditions has a much lower effect on all properties except modulus.
32	Durability of fiber reinforced composite wrapping for the rehabilitation of concrete piers	Kshirsagar S, Lopez-Anido R & Gupta RK.	Proceedings of CDCC 98, Montreal, 1998: 117–128	Evaluate durability of GFRP-wrapped cylinders to alkaline solution at room temperature or elevated temperature	Exposure of GFRP-wrapped cylinders to alkaline solution (or water) at room temperatures had no effect. However at elevated temperature (65.5^{o}C), exposure to alkaline solution or water was detrimental. Strength reductions were in excess of 25% after 1000 hours of exposure.
33	Strengthening of reinforced concrete with epoxy-bonded carbon-fiber plastics	Kaiser H	Ph.D. thesis, ETH, Zurich, Switzerland	Temperature effect on concrete beams	The temperature effect over 100 freeze-thaw cycles from +20 ^{o}C to -25 ^{o}C on concrete beams strengthened with CFRP has negative influence on the flexural capacity.
34	Durability characteristics of concrete columns confined with advanced composite materials	Toutanji HA	Composite Structures 44 (1999):155-161	To evaluate the durability performance of concrete columns confined with FRP sheets	Confinement of concrete cylinders with FRP sheets improves the compressive strength and ductility. Type A & B epoxy matrices produced similar results for the unconditioned specimens. Specimens confined with GFRP sheets using Type A epoxy exhibited a loss in strength due to wet/dry conditioning. Using Type B epoxy resulted in an insignificant loss in strength. Wet/dry exposure produced no loss in ductility in specimens wrapped with CFRP sheets, whereas those wrapped with GFRP sheets exhibited a reduction in ductility.

No.	Title	Author / Authors	Source	Objective	Outcome
35	Durability of composite material reinforcement	Chajes MJ, Mertz DR, Thomson JR	Proceedings of the Third Materials Engineering Conference, ASCE, Dallas, TX, 1994:598-605	Durability of several composite systems externally bonded to concrete beams exposed to freeze-thaw and wet-dry cycles	A loss in flexural strength due to a degradation of the bond between the concrete and the external composite reinforcement was observed.
36	Durability of fiber glass pretensioned beams	Sen R, Marseal D, Shahawy MR	ACI Structural Journal 1993: 90(5):525-533	Durability of S-2 glass/epoxy pretensioned beams subjected to wet/dry cycles simulating tidal effects	A complete loss in the effectiveness of the fiberglass strands exposed to wet/dry cycles after an average of 6 months for precracked beams and 15 months for the uncracked beams. This extensive damage in the fiberglass pretensioned beams led to an unsatisfactory level of strength loss.
37	Experimental behavior of concrete beams with externally bonded carbon fiber tow sheets	Javed S, Kumar SV, GangaRao VS	Presented at the 51st Annual Meeting of SPI/CI Conference & Exposition, February 1996	The effect of acidic and alkaline conditioning, under varying and constant temperature, on the strength and stiffness of concrete beams wrapped with carbon fiber sheets	The bond shear strength of samples exposed to environmental conditioning decreased with respect to the unconditioned samples. The percentage decrease was 17% for samples in an acidic condition, 24% for samples in an alkaline condition, and 29% for samples exposed to hygrothermal condition.
38	Durability studies on concrete columns encased in PVC-FRP composite tubes	Toutanji H, Saafi M	Composite Structures 54 (2001): 27-35	Performance of hybrid concrete columns subjected to room temperature, freeze-thaw, and wet-dry conditions	Concrete confined with PVC-CFRP showed no loss in strength due to 200 and 400 cycles of wet/dry or freeze/thaw. Concrete confined with PVC-AFRP showed no reduction in strength due to 200 cycles of wet/dry or freeze/thaw. Concrete confined with PVC-GFRP exhibited small reductions in strength. Concrete confined with PVC-GFRP and PVC-AFRP showed a loss in strength and axial strain due to 400 freeze/thaw cycles. Freeze/thaw exposure exhibited much more catastrophic failure than room temperature or wet/dry condition.

I-A.1.3.5 Case Studies

The high strength to weight ratio and corrosion resistant properties of FRP materials have resulted in their preferential use for structural rehabilitation. Composite plates and fabrics are being successfully used to retrofit masonry, wood and concrete beams, columns, buildings, bridges and other structures damaged/weakened by impact, earthquake or aging. Individual project case histories are presented in this section.

Case 1: State Route 99 Alaskan Way Viaduct
(Watson Bowman Acme, http://www.wbacorp.com/downloads/Bridge/)

The 48-year-old State Route 99 Alaskan Way Viaduct suffered damage as a result of the February 28, 2001 Nisqually earthquake of magnitude 6.8 Richter. MBrace carbon fiber wrap was used to reinforce the crossbeams and edge beams of the four damaged piers (Project supervised by Mr. Tysl while working at Watson Bowman Acme). The longitudinally oriented carbon fiber plies were bonded to the underside of the crossbeams to augment positive bending moment capacities. Transversely oriented U-shaped plies were bonded to both the crossbeams and edge beams to augment their vertical shear capacities (see Fig. I.A.1.9).

Fig. I.A.1.9 Repair of earthquake damaged Alaskan Way Viaduct.

Case 2: Sins Covered Timber Bridge in Switzerland
(FHWA study tour for advanced composites in bridges in Europe and Japan, October 1997, http://ntl.bts.gov/DOCS/advcom3a.html)

Illustrated in Fig. I.A.1.10 is the Sinns covered wooden arch bridge in Switzerland, built in 1807; this bridge was successfully rehabilitated in 1992 by externally reinforcing with CFRP plates bonded to the transverse crossbeams.

Fig. I.A.1.10 Sinns covered timber bridge.

Case 3: Bridge in Florida
(North Carolina State University)

Figure I.A.1.11 shows the underside of a bridge in Florida damaged by vehicles that exceeded the height limit (NC State Univ. web site). This retrofit project was led by Prof. Mirmiran with collaboration among the NC State Univ., SDR Engineering Consultants, Univ. of Missouri-Rolla and Univ. of California-San Diego. The bridge was externally reinforced with CFRP, and was regularly tested to evaluate the performance of the strengthened components, indicating enhanced performance of the bridge structure.

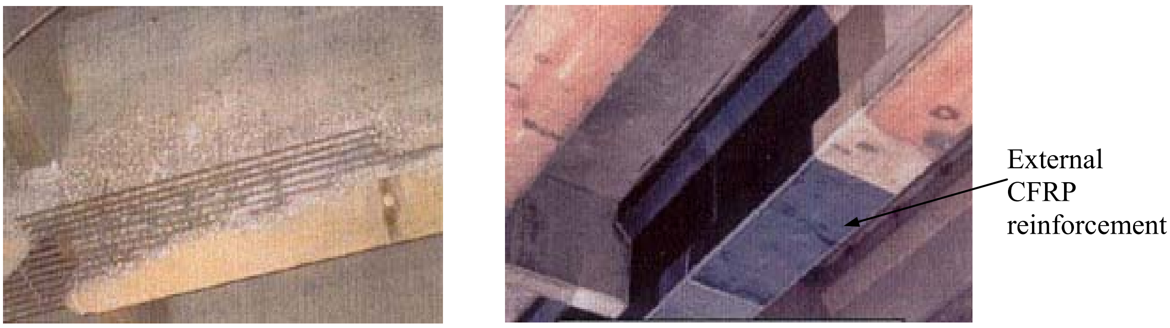


Fig. I.A.1.11 Repair of bridge in Florida with CFRP.

Case 4: South Temple Bridge in Salt Lake City
(University of Utah, http://www.civil.utah.edu/~chris/research/retrofitofRCC.pdf)

The South Temple Bridge in Salt Lake City was damaged due to earthquake; the bridge bent was retrofitted using carbon FRP (see Fig. I.A.1.12). The in-situ tests showed that externally bonded CFRP composites can greatly enhance the ductility of the bridge bents. In addition, the cap beam-column joint shear capacity was enhanced, overall damage was controlled, and a residual strength was left to support the dead load. A 35% increase in shear strength was observed for joints retrofitted with CFRP. The in-situ tests proved that the design of the composite was successful and provided strong evidence that the retrofit using CFRP composites is a viable option for improving the seismic performance of reinforced concrete bridges (Univ. of Utah website).

External CFRP
reinforcement

Fig. I.A.1.12 South Temple Bridge.

Case 5: I-96 Overpass
(www.egr.msu.edu/cee/research/infrastructure/PROJ_PCJCR.shtml)

The columns shown in Figure I.A.1.13 are located on Lansing Road in Lansing, Michigan under the I-96 overpass (Bridge ID S09 and S10 of 23152). These columns had considerable surface spalling and the reinforcement was exposed at several locations. After superficial repairs to the column surface, two columns were wrapped with two layers of carbon fiber sheets, and two were wrapped with three layers of glass fiber sheets. The wrapped columns showed significant improvement in strength.

Fig. I.A.1.13 Column repair of bridge over I-96.

Case 6: Houghton Highway Bridge
(University of South Australia,
http://business2.unisa.edu.au/prres/Proceedings/Proceedings2003/Humphreys_Extending_service_life_buidings_infrustracture_fibre_composites.pdf)

The Houghton Highway Bridge is a dual carriageway bridge in Australia linking Queensland's capital Brisbane with the northern shire of Redcliffe. A routine inspection carried out in 1991 identified deterioration of the prestressed concrete piles due to exposure to marine water. There was evident internal cracking and spalling of concrete thus exposing the reinforcing steel to marine environment. In 2000, these piles were externally reinforced with carbon FRP (see Figure I.A.1.14).

Fig. I.A.1.14 Houghton Highway Bridge.

Case 7: Freedom Tower in Miami
(Structural Group 2002,
http://www.structural.net/News/Media_coverage/ci2410powers.pdf)

Beam repairs were required throughout the Freedom Tower structure, including concrete encased I-beams on certain floors in the office tower portion (Fig. I.A.1.15). All delaminated and spalled concrete was removed from the beams. Beams supporting the library level were externally bonded with a composite carbon fiber-reinforced plastic strengthening system to withstand the additional load-bearing requirements. The thin cross-section of CFRP strengthening system made it possible to conceal their presence and preserve the beauty of the existing structure. Thus, the Freedom Tower has been successfully restored to its ancient glory with minimum possible alteration in its external appearance.

(a) (b) (c)

Fig. I.A.1.15 (a)Freedom Tower, (b)Deteriorated beam & slab, (c)Application of CFRP.

Case 8: Farm-to-Market Bridge in Ward County in the Odessa District of Texas
(Bradberry and Wallace 2003)

An external beam of the Farm-to-Market 1927 bridge was severely damaged by an over-height vehicle on January 17, 2002. The beam's web and the bottom flange near the middle of the span between two diaphragms were fractured into several pieces as shown in Figures I.A.1.16 and I.A.1.17. The maintenance engineer proposed one layer of CFRP composite in addition to the normal repair method to restore the shear strength and the integrity of the beam. The direction of the longitudinal fiber was perpendicular to the longitudinal beam axis (vertical to the web) because the CFRP was intended for shear strengthening. Patching of the concrete and the epoxy injection took four days; CFRP composite installation took one day. The total cost was $47,000, which included $22,000 for concrete repair and $25,000 for CFRP composite installation. After completion of all repair work, all lanes under the bridge were opened to traffic in April 2002 (see Fig. I.A.1.18).

Fig. I.A.1.16 Exterior Girder Bottom Flange after Impact with Over-height Vehicle/Payload.

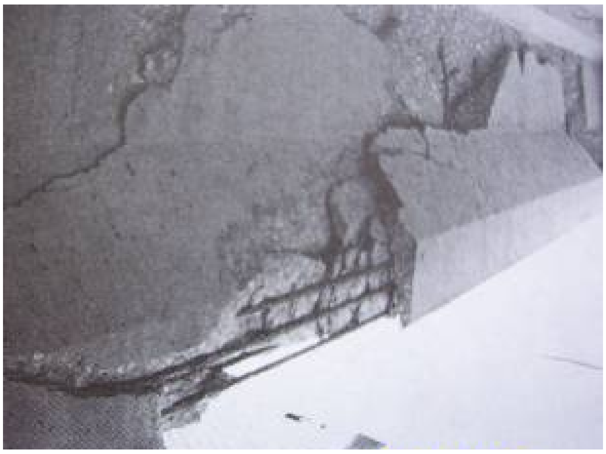


Fig. I.A.1.17 Interior Face of External Girder after Impact with Over-height Vehicle/Payload.

Fig. I.A.1.18 Damaged External Girder after Completed Repair Work.

Case 9: Jacques Lodge Pedestrian Bridge in Calgary, Alberta, Canada
(Hutchinson et al. 2003)

The Jacques Lodge Pedestrian Bridge in Calgary, Alberta, consists of a precast concrete “drop-in” girder supported by a cantilevered girder at each end. As shown in Fig. I.A.1.19, the concrete corbels supporting the main girder exhibited cracking due to unexpected friction forces and a lack of sufficient internal diagonal steel reinforcement. The direction of the fibers in the applied sheets was consistent with the ordination of the deficient reinforcing steel. The configuration of the CFRP sheets is shown in Fig. I.A.1.20 (a). A single horizontal layer and a diagonal layer of CFRP sheets were required for each side of the corbel, as shown in Fig. I.A.1.20 (a). In order to repair the concrete corbels, the entire main span was jacked up. After cleaning and preparing the concrete surface, Replark™ CFRP sheets were applied as shown in Fig. I.A.1.20 (a). After curing of the epoxy impregnating the CFRP sheets, a protective coating was applied, and the jacks supporting the main span were removed, as shown in Fig. I.A.1.20 (b). Repair of the bridge was completed in 6 weeks at a total cost of $20,000.

Case 10: Bridge A5657 over Gasconade River, South of Dixon, MO.
(Parretti et al. 2003)

The original ultimate flexural capacity of an accidentally impact-damaged PC girder of Bridge A5657 (see Fig. I.A.1.21) located on Route 28 over the Gasconade River, South of Dixon MO was restored. Two prestressing tendons in the central girder of the North span of the bridge were fractured due to the impact, as shown in Fig. I.A.1.22. Figures I.A.1.23 and I.A.1.24 show the installation of the FRP laminates and the U-wrap used to hold in place the flexural reinforcement, respectively.

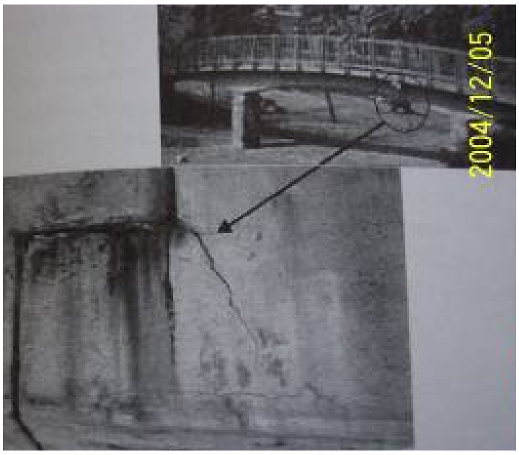


Fig.I.A.1.19 Jacques Lodge Bridge Corbel Support Prior to Repair.

Fig. I.A.1.20 (a) Application of CFRP Sheets.

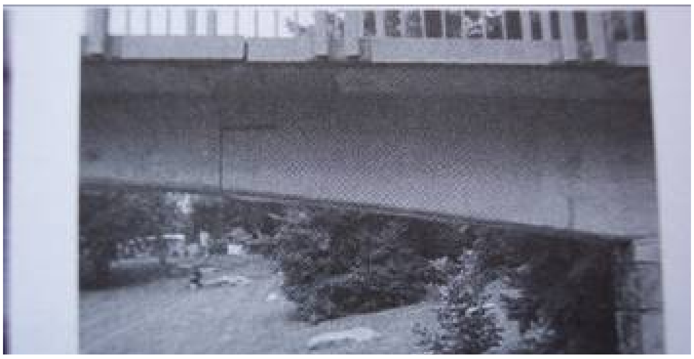


Fig. I.A.1.20 (b) Bridge after Repair.

Fig. I.A.1.21 Bridge A5657.

Fig. I.A.1.22 Damaged Area in the Girder.

Fig. I.A.1.23 Installation of the Flexural Reinforcement.

Fig. I.A.1.24 U-Wraps Installation.

Case 11: Highway 10 Overpass Columns, Quebec
(Neale and Labossiere 1998)

In August 1996, nine of twelve corrosion-damaged circular columns (Fig. I.A.1.25) of an overpass located on Highway 10 in Quebec were repaired in three weeks, using FRP composites. Damage to the 6-m-high, 760-mm diameter reinforced columns was the result of splashing of salt contaminated by vehicular traffic in winter. Innovative fiber optic sensors were used for instrumenting the columns. Four of the columns were wrapped with carbon and five with glass fiber (Fig. I.A.1.26). Fiber optic sensors were installed on four of the rehabilitated columns. Deformations of the columns due to temperature variation, effects of loading and corrosion of reinforcement were monitored on a regular basis. The performance of the columns since repair has been excellent.

Fig. I.A.1.25 View of corroded columns, Highway 10 overpass, Quebec, Canada.

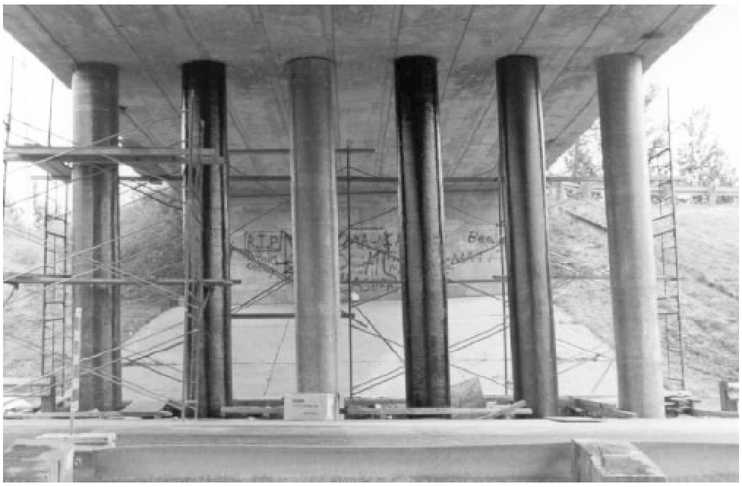


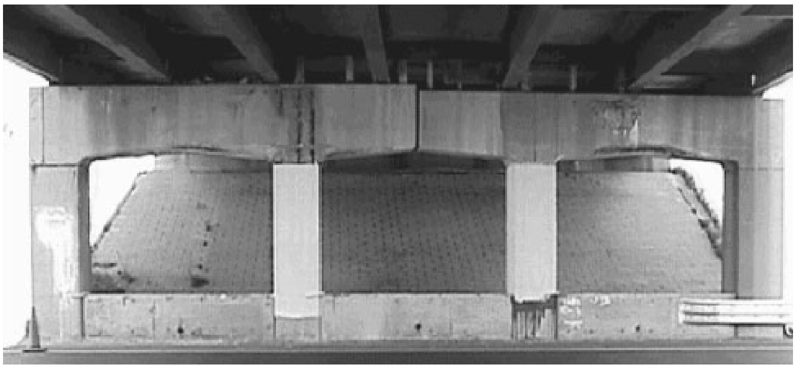
Fig. I.A.1.26 View of wrapped columns, Highway 10 overpass, Quebec, Canada**.**

Case 12: Michigan DOT
(Harichandran and Baiyasi2000)

The Michigan Department of Transportation (MDOT) used FRP to repair four corrosion-damaged columns on Lansing Road, Lansing, MI under the I-96 overpass. The repaired reinforced columns were 81×81 cm. Corrosion damage was relatively minor, comprising spalls and corner crack (Fig. I.A.1.27 (a)). All columns were instrumented on one face with corrosion probes manufactured by Rohrback Cosasco Systems, Inc. The probes were located at the same level of the existing reinforcement about 2m above the road surface (Fig. I.A.1.27 (b)). Superficial repairs were first carried out with patching material to which chloride had been added to match the pre-existing chloride in the columns. Two columns were wrapped with two layers of unidirectional carbon fiber sheets (MBRACE CF-130) and two others with three layers of unidirectional glass (FYFE SHE 51, TYFO S). Two additional unwrapped rectangular columns 96.5×81 cm were retained as controls. The repairs were completed in July 1999 (Fig. I.A.1.27 (c)).

Estimated cost savings over conventional chip and patch repair used by MDOT ranged from \$75–300/m^2 (of the repaired column surface) for glass fiber to \$140–365m^2 for carbon. Estimated material costs for carbon were about 40% higher, but required one fewer layer because of its higher strength and stiffness. After 10 months of monitoring, no significant corrosion activity was detected. The original plan was to monitor the corrosion rate for 10 years, but this is not being carried out. However, the performance of the columns continues to be satisfactory, though some increase in delamination that occurred during initial installation was observed.

(a) (b)



(c)

Fig. I.A.1.27 (a) Corrosion damage in East Bound Pier; (b) Preparation for installing corrosion probes and wiring; (c) View of repaired columns (left) carbon and glass (right).

The above case studies show the versatility, efficiency, and wide spread use of externally bonded FRP materials for rehabilitation of deteriorated concrete structures. However, since most of these repairs were carried out in the past ten years, the long-term durability of such structures is not very well established and will require further investigations.

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Sub-Task I-A.2: Surveys and/or Interviews

To evaluate the feasibility of surface bonded FRP technology (referred to as “technology” thereafter) for potential applications to District-3 concrete T-beam bridges, a survey questionnaire was designed, and is given in Appendix A. The survey was directed to state DOTs, specialized contractors, suppliers and design engineers, as well as academia who have been involved in the planning and/or application of FRP technology in the field. The implementation of the survey was carried out between April 2005 and July 2005. The survey (see Appendix A) included a brief explanation of the purpose and three specific areas of interest, as follows:

1. Applications of surface bonded FRP technology: To gather the information pertaining to the applications experience with surface bonded FRP technology. In particular, the methods used for determining applicability of the technology, level of damage and specific conditions, cost effectiveness and degree of satisfaction, and finally the opinion about best conditions and limitations for technology implementation.
2. Implementation protocols for surface bonded FRP technology: To gather the information pertaining to the implementation protocols being used currently by state DOTs in collaboration with relevant industries and consultants. In particular, the management and responsibilities of the different entities engaged, including researchers, technology suppliers, consulting and/or in-house design engineers, contractors, and owners.
3. Evaluations and assessments of surface bonded FRP technology: To gather the information pertaining to evaluations and assessments of the technology. In particular, practices used for pre-repair conditions, materials and methods during construction, inspections and tests after completion, subsequent long-term studies, and finally, overall degree of satisfaction and opinion on best practices for evaluations and assessments.

The survey was designed by the WVU researchers in collaboration with Steve Tysl of Proto Composites. The implementation and data gathering was accomplished by Don Matzzie of Linare Consulting.

Table I.A.2.1 presents a list of all potential interviewees, including 29 state DOTs, 4 academics, and 2 contractors/suppliers. Table I.A.2.2 presents a list of all respondents in the survey from different regions of the U.S., including 13 from state DOTs and one from academia. As for the effectiveness of gathering the desired information, Linare Consulting stated that their experience with similar past efforts indicated that reliance on mail to get survey instruments into the hands of targets and returned is not reliable. Thus the targets were contacted in advance by phone and/or e-mail, and transmitted the survey forms by e-mail or fax. Linare Consulting then proceeded to re-contacting the targets by phone to prompt return of the completed forms by fax or e-mail, or took answers to the questions over the phone.

It should be emphasized here that the lack of response from any state DOT, contractor, supplier, or academia may not necessarily mean their lack of FRP application experiences. While the low rate of return understandably makes the survey less than "scientific," it may still serve as an informative gauging or "polling" tool of the current state of surface bonded FRP technology. The list of respondents provides a representative cross section of the current practice of applications of surface bonded FRP technology, implementation protocols, and evaluations and assessments. The majority of the respondents are from state DOTs. Only one respondent is from academia. No respondent is from contractors/suppliers.

Table I.A.2.1 List of Potential Interviewees

Affiliate	Potential Interviewees	E-mail	Phone
CA DOT	Charles Sikorsky	charles_sikorsky@dot.ca.us	916-227-8759
CalTrans	Rick Carter	richard_r_carter@dot.ca.gov	916-227-8625
CalTrans	Jim Gutierrez	jim_gutierrez@dot.ca.gov	916-227-8256
CO DOT	Mike Mohsemi		303-512-4300
GA DOT	Paul Lyle		404-463-2037
IA DOT	Scott Neubauer		515-239-1290
IA DOT Dist 1 Ames	Norb Kottlers		515-239-1564
ID DOT	Matt Farrar		208-334-8538
IL DOT	Ton Domagalski	domagalaskitj@dot.il.gov	217-782-2125
IL DOT	Gary Kowalski	kowalskigm@dot.il.gov	217-785-2914
IL DOT	Joe Lenzini	lenzinij@dot.il.gov	618-549-2171
IL DOT Main Springfield	Salah Khayat		217-785-2923
LA DOT	Walid Alaywan	walaywan@dotd.louisiana.gov	225-767-9106
MI DOT	Doug Needham		517-335-2229
MI DOT	Roger Till		517-322-5682
MO DOT	Rick Pilcher	Richard.pilcher@modot.mo.gov	573-526-4328
NC State	Sami Rizkalla	sami-rizkalla@ncsu.edu	919-513-1733
NY DOT (ret.)	Vince Kazakavich		518-346-3422
NYS DOT	Osman Hag-Elsafi	ohag-elsafi@dot.state.ny.us	518-457-5826
NYS Thruway	Dan Marcus		315-437-2741
ODOT Central	Scott Seeley		614-644-5754
ODOT Dist 12 Cleve	Michael Malloy		216-581-2333
OR DOT	Mike Dunning		503-986-3059
OR DOT	Robert Goodrich		503-986-3311
OR DOT (former)	Guido Portier		503-361-8635
Queen Univ.	Mark Green	greenm@civil.queensu.ca	
Structural Preservation Systems	Jay Thomas	jthomas@structural.net	877-669-4875
U of M-R	Nestore Galati	galati@umr.edu	573-341-6223
UCSD	Visrasp Karbhari	vkarbhari@ncsd.edu	858-534-6470
UDOT Research	Daniel Hsiao		801-965-4638
UDOT Structures	Boyd Wheeler		801-964-4456
VT DOT	Craig Graham	craig.graham@state.vt.us	802-828-2561
WA DOT	Geoffrey Swett		360-705-7157
WI DOT	James Oettinger, PE		608-246-3879
WIDOT	Dale Weber		920-492-7161

Table I.A.2.2 List of Respondents

Affiliate	Potential Interviewees	E-mail	Phone
CalTans	Charles Sikorsky	charles_sikorsky@dot.ca.us	916-227-8759
CalTrans	Rick Carter	richard_r_carter@dot.ca.gov	916-227-8625
CalTrans	Jim Gutierrez	jim_gutierrez@dot.ca.gov	916-227-8256
CO DOT	Mike Mohsemi		303-512-4300
IA DOT	Scott Neubauer		515-239-1290
ID DOT	Matt Farrar		208-334-8538
IL DOT	Ton Domagalski	domagalaskitj@dot.il.gov	217-782-2125
IL DOT	Joe Lenzini	lenzinij@dot.il.gov	618-549-2171
LA DOT	Walid Alaywan	walaywan@dotd.louisiana.gov	225-767-9106
MI DOT	Roger Till		517-322-5682
MO DOT	Rick Pilcher	Richard.pilcher@modot.mo.gov	573-526-4328
NY DOT	Osman Hag-Elsafi	ohag-elsafi@dot.state.ny.us	518-457-5826
U of M-R	Nestore Galati	galati@umr.edu	573-341-6223
UTDOT Research	Daniel Hsiao		801-965-4638

Section 2 - Task I-B: Analysis of Findings

A number of practical issues and findings relevant to FRP repair and retrofit of concrete structures are discussed in this section, on the basis of all of the information gathered through the literature review and the survey results in Task I-A. This report for Task I-B – Analysis of Findings, includes the synthesis from the literature review (Sub-Task I-A.1 – Literature Search), and the analysis of the survey results (Sub-Task I-A.2 – Surveys and/or Interviews). Both of these are then combined into one comprehensive summary synthesis of findings.

Sub-Task I-B.1: Analysis from Literature Search

The information from Sub-Task I-A.1 – Literature Search is analyzed in this section, in order to discuss: (1) assessment of relevant and pertinent applicable situations for surface-bonded FRP technology; (2) implementation of technology for concrete repair by industry; (3) effectiveness, life-span and durability in relation to concrete deterioration; (4) cost-effectiveness of the technology; and, (5) limitations for application of the technology.

I-B.1.1 Applicable situations for FRP technology

FRP composites used in new bridge construction and rehabilitation have provided bridge engineers with innovative solutions for today's infrastructure problems. The beneficial characteristics of composites such as light weight, high stiffness-to-weight and strength-to-weight ratios, and potentially high resistance to environmental degradation contribute to easy transportation, offsite construction, modular assembly, rapid installation, and durability, resulting in lower life-cycle costs. FRP composites are increasingly being considered for use in civil infrastructure applications. The emerging field of renewal engineering may best describe the role of FRP composites in civil engineering. The renewal of the structural inventory, which is depicted in Fig. I.B.1.1, can be divided into (1) rehabilitation, including the applications towards repair, strengthening and retrofit of structures; and (2) new construction with all FRP solutions or new composite FRP/concrete systems (Van Den Einde et al. 2003).

Although many countries had built new structures using FRP composites, the structures were all in one-of-a-kind demonstration projects to showcase the technology. The most promise for the use of FRP composites in civil engineering was found in structural rehabilitation. Within the scope of rehabilitation of concrete structures, it is essential to differentiate between repair, strengthening and retrofit, terms which are often erroneously used interchangeably, but in fact refer to three different structural conditions (Karbhari and Zhao 2000). In **'repairing'** a structure, the composite material is used to fix a structural or functional deficiency such as a crack or a severely degraded structural component. In contrast, the **'strengthening'** of structures is specific to those cases wherein the addition or application of the composite would enhance the existing designed performance level, as would be the case in attempting to increase the load rating (or capacity) of a bridge deck through the application of composites to the deck soffit. Carbon fiber-reinforced polymers are commonly used in strengthening projects because CFRPs offer the best strength, stiffness, and durability characteristics. The term **'retrofit'** is specifically used as related to the seismic upgrade of facilities, such as in the case of the use of composite jackets for the confinement of columns. Seismic retrofit also requires consideration of structural ductility, in addition to strength and stiffness considerations.

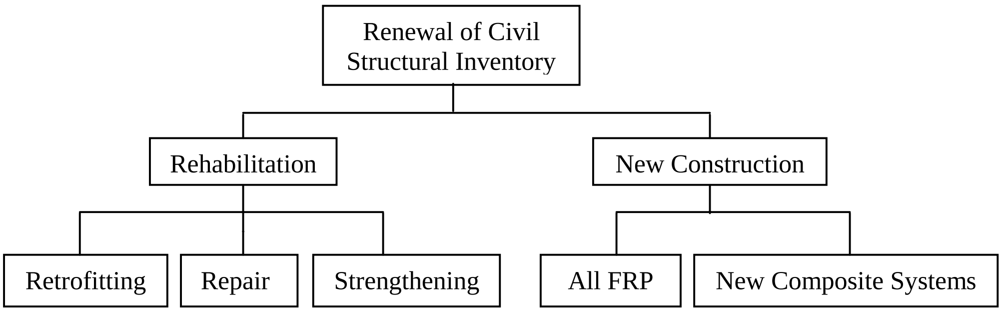

Fig. I.B.1.1 Use of FRP composites in civil engineering (Van Den Einde et al. 2003).

FRP as reinforcement in concrete essentially provides strength where concrete is weakest – in tension. Recent FRP rehabilitation projects have had a broad range of varied characteristics, and have been carried out in regions encompassing a wide variety of environmental conditions. They range from bridges to ground anchors and from architectural engineering or building applications to marine structures. Industrial facilities, dams, underground tanks, or drainage systems account for the remainder. The reported laboratory and field test results are quite encouraging not only for pristine FRP-concrete specimens, but also for pre-cracked members, and samples subjected to some form of environmental exposure. From the information gathered through the literature review, the applicable situations of FRP composites can be summarized as:

- Using externally bonded FRP laminates on *beam, slab and deck soffits* to provide additional flexural strength

- Using externally bonded FRP laminates on *sides of beams* to provide additional shear strength
- Wrapping FRP around *bridge columns* to provide confinement and additional ductility (a primary concern in seismic upgrades)
- Rehabilitating *deteriorated columns and piers* by CFRP/GFRP wraps
- Strengthening concrete and masonry *walls* to better resist seismic and wind loads
- Strengthening *silos*, *tanks*, and concrete *pipes* to resist higher pressure or restore loss of load carrying capacity due to corrosion of the prestressing wires
- Repairing *bridge elements corroded due to salt water* runoff from faulty expansion joints or salt spray from passing vehicular traffic
- Underwater repair of *corrosion damaged piles* (reinforced and prestressed)
- Strengthening *T-beams* of parking garage to increase design load
- Using a wet lay-up FRP system to *strengthen bridge girder* for flexure and shear
- Repairing of reinforced concrete *beams and columns* in which the steel reinforcement has been damaged by *corrosion* to improve the strength and serviceability
- Strengthening *cracked concrete beams* for improving flexural and shear capacities
- Strengthening multi-purpose convention centers, office buildings, and other *facilities*
- Applying FRP laminates to repair *aging RC T-beams* with efflorescence, freeze-thaw cracking and concrete delamination due to moisture and salt infiltration
- Strengthening concrete *beams* with a triaxial ductile FRP system to offer strength, stiffness, and ductility

FRP plate bonding technique is now popular in the in-situ enhancing of concrete beams in bridges that have either partially lost their embedded flexural steel through corrosion or that need upgrading due to increased load requirements. Many bridges to which the technique is applied are single span and simply supported (Sebastian 2001).

The use of FRP products can provide the following benefits:

- Longer structural service life
- Good fatigue resistance and high strength-to-weight ratio, leading to higher durability
- Cost effective on an installed cost basis
- Reduce maintenance costs due to resistance to deicing salts and other corrosive agents

- Reduce field installation time due to engineered system packaging and light weight
- Reduce traffic delays due to faster construction
- Enhanced safety as minimal equipment and formwork is needed, and long traffic delays are eliminated
- Increase system reliability due to pre-engineered systems and corrosion resistance
- Allow greater vehicular load on the same understructure due to weight reduction
- Thin strengthening layers that do not change the dimension of the existing structure, especially for a historic structure which needs to maintain its original appearance, and can also be combined with thin concrete overlays or surface protecting materials
- Prestressing possibilities give a higher utilization of the strengthening product, reduce existing cracks, increase the yield load of the existing steel reinforcement, and increase the shear capacity of concrete structures
- The possibility to optimize the FRP materials in the direction most needed is a benefit for design

From the wide range of applications that have been successfully carried out in all regions of the world and based on the studies carried out over the past years, it can be concluded that FRPs have found a particularly attractive niche in applications involving strengthening, repair and retrofitting of concrete structures. It is envisaged that in time to come, this technology will be a widely accepted practice in the construction industry and its applications will not be limited to those that have been described above.

I-B.1.2 Implementation of technology for concrete repair by industry

The information we collected and synthesized from the literature and the survey has permitted us to analyze in this section the technical and organizational approaches that industry and state DOTs have tried and adopted to implement this technology.

Substrate Repair and Surface Preparation (ACI 440.2R-02; Mirmiran, et al. 2004)

The behavior of concrete members strengthened or retrofitted with FRP systems is highly dependent on a sound concrete substrate and proper preparation and profiling of the concrete surface. An improperly prepared surface can result in debonding or delamination of the FRP system before achieving the design load transfer. Specific guidelines for a particular FRP system should be obtained from the FRP system manufacturer.

The issues for substrate repair include types of defects and methods of repair for the concrete substrate and the internal reinforcement. The work consists of several steps, including removal of defective concrete, repair of defective reinforcement, restoration of concrete cross section, and injection of cracks.

It is imperative that the damaged structure or the structure with defects be properly prepared prior to the application of any FRP repair system. Improper treatment of concrete and the exposed reinforcement can lead to failure of the repair system. Any *loose concrete* remaining in the damaged region must be removed, leaving the member with sound concrete. Externally bonded FRP systems should not be applied to concrete substrates suspected of containing *corroded reinforcing steel*. The expansive forces associated with the corrosion process could compromise the structural integrity of the externally applied FRP system. Any corroded reinforcement steel must be repaired and treated (sandblasting of corroded steel). Improper *water-proofing and splice details* can allow further corrosion of the internal reinforcement, leading to loss of capacity and ductility. Damaged reinforcement may need to be spliced. *Cracks* wider than 0.3 mm should be pressure injected with epoxy in accordance with ACI 224.1R. Smaller cracks exposed to aggressive environments may require resin injection or sealing to prevent corrosion of existing steel reinforcement. *Restoration of a concrete section* to its original shape may require small patching or considerable concreting with formwork. The quality and strength of the patching material and its bond with the existing concrete are important considerations. The bond may be enhanced with mechanical anchorage in the repaired region.

Surface preparation of the substrate is essential in achieving a good bond with the FRP repair system. The FRP repair applications are often categorized into two types: bond critical and contact critical. *Bond-critical* applications, such as flexural or shear strengthening of beams, slabs, columns, or walls, require an adhesive bond between the

FRP system and the concrete substrate. *Contact-critical* applications, such as confinement of columns, only require intimate contact between the FRP system and the concrete substrate. Even though bonding may not be structurally necessary in the confinement of columns, it should be promoted for durability purposes. Promoting bonding between FRP and concrete on all projects and for all surfaces can lead only to better construction practice at this early stage of development of the FRP technology.

The surface must be cleaned of all laitance, dusts, dirt, oil, curing compound, existing coatings by appropriate means. It must also be made free of moisture and frost before installing the FRP repair system. Surface irregularities affect the bond between FRP and concrete. They also may result in localized stress concentration. Such irregularities should be ground smooth within acceptable tolerances (CSP 3 as defined by the ICRI-surface-profile chips). Bug holes and voids should be filled with epoxy putty. Rounding the corners reduces stress concentration and results in an improved bond between the FRP and the concrete surface. The minimum radius should be 1/2 in. (13 mm). In applications involving confinement of structural concrete members, surface preparation should promote continuous intimate contact between the concrete surface and the FRP system. Surfaces to be wrapped should, at a minimum, be flat or convex to promote proper loading of the FRP system. Large voids in the surface should be patched with a repair material compatible with the existing concrete.

FRP Repair Systems

There are three common FRP system forms suitable for the strengthening of structural members (ACI 440.2R-02; Mirmiran et al. 2004). These are *wet layup systems, prepreg systems, and precured systems*. Other FRP systems include *surface mounted, FRP rigid rod* and *flexible strand or cable.* Near surface mounted (NSM) FRP repair systems involve inserting and bonding FRP strips or rods into precut grooves. NSM is a promising technique for increasing the flexural and shear strength of deficient reinforced concrete members. When compared to externally-bonded reinforcing technique, NSM technique shows several advantages, namely: lesser susceptibility to the peeling phenomenon; extra protection against fire and mechanical and environmental damage effects; easier and faster installation (Sena Cruz and Barros 2003).

FRP systems react differently to the environmental conditions and vary in mechanical properties. Issues related to the effects of environmental conditions on different FRP systems are shown in Table I.B.1.1.

Table I.B.1.1 Environmental consideration for different FRP systems (Mirmiran et al. 2004).

Consideration	Carbon	Glass	Aramid
Alkalinity/acidity exposure	Highly resistant	Not tolerant	Not tolerant
Thermal expansion	Near zero, may cause high bond stress	Similar to concrete	Near zero, may cause high bond stress
Electrical conductivity	High	Excellent insulator	Excellent insulator
Impact tolerance	Low	High	High
Creep rupture and fatigue	High resistance	Low resistance	Low resistance

The primary issues for FRP installation include application of adhesives, FRP sheets or precured laminates, and protective coatings. Resins must be mixed at appropriate environmental conditions and must be used within their pot life. Application of the resin must be such that air voids are not present. Alignment of fiber sheets or precured laminates and any necessary overlaps in multiple layers also affect the performance of the FRP system.

Wet lay-up and precured FRP systems may be prestressed to improve their performance. Prestressing may be developed using active end anchorages in linear applications for beams, or using pressure grouting in circular application for active confinement of concrete columns. Active confinement is not recommended for glass FRP systems because the susceptibility of glass FRP systems to creep rupture. Moreover, the prestrain in carbon FRP systems should be limited to 50% of the ultimate strain due to damage tolerance concerns with unidirectional carbon FRP.

Methods of Application of External FRP for Strengthening

In general, composites can be applied in three ways as described in Table I.B.1.2, of which the first two are the most widely used. It should be noted that although the wet lay-up process affords significant flexibility for work on site, and is still by far the most commonly used process in the field, there may be significant advantages, technical and practical, in the use of prefabricated and hence presumably standardized strips and plates, which are adhesively bonded to the concrete substrate.

Table I.B.1.2 Methods of application of external composite reinforcement for strengthening (Karbhari and Seible 2000).

Procedure	Description	Time/Issues
Adhesive bonding	Composite strip/panel/plate is pre-fabricated and then bonded onto the concrete substrate using an adhesive under pressure	• Very quick application • Good quality control
Wet lay-up	Resin is applied to the concrete substrate and layers of fabric are then impregnated in place using rollers and/or squeegees (or a pre-impregnated wet layer of fabric is squeezed on). The composite and bond are formed at the same time	• Slower and needs more setup • Ambient cure effects • Waviness/wrinkling of fiber • Non-uniform wetout and/or compaction
Resin infusion	Reinforcing fabric is placed over the area under consideration and the entire area is encapsulated in a vacuum bag. Resin is infused under vacuum. In a variant the outer layer of fabric in contact with the bag is partially cured prior to placement in order to get a good surface	• Far slower with need for significant setup • Ambient cure effects • Dry spots

Plating Techniques

There is an enormous variety of plating techniques that are available to the engineer. Plates can be bonded to the sides of beams, to their tension faces and to their compression faces (see Fig. I.B.1.2). These plates can be bolted or they can be adhesively bonded. Furthermore, these plating techniques can be used to improve the shear and flexural capacities as well as the serviceability requirements (Oehlers 2001). Recently, a new strengthening method, termed the fastened method, has been developed where the strengthening strip is entirely mechanically attached to the concrete surface using multiple small, distributed powder-actuated fasteners without any bonding (Lamanna et al. 2004). Figure I.B.1.3 shows the fasteners spacing on the surface of a cracked concrete beam. The use of multiple small fasteners, as opposed to large-diameter bolts, distributes the load more evenly over the composite strip and does not cause catastrophic failure due to excessive stress concentrations at the holes in the composite strip.

FRP strengthening strips attached to reinforced concrete beams with powder-actuated fasteners were as effective as the traditional method of bonding the strips to beams. When long fasteners driven into predrilled holes were used, the fastened method provided increased ductility over the bonded method.

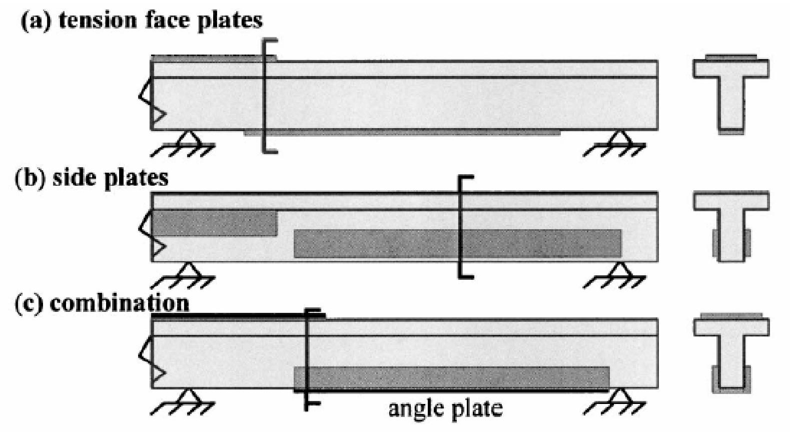

Fig. I.B.1.2 Plate positions (Oehlers 2001).

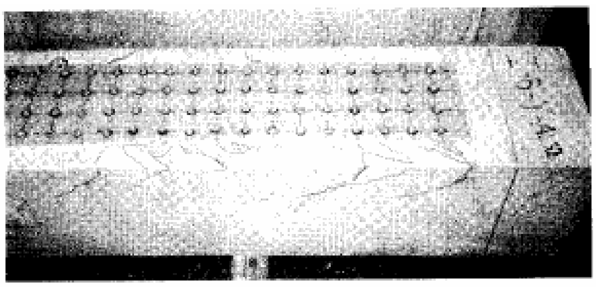

Fig.I.B.1.3 Fasteners spacing on beam surface (Lamanna et al. 2004).

Tension face plates (Fig. I.B.1.2 (a)) are mechanically efficient and accomplish the highest increase in flexural strength and stiffness; however, the addition of tension face plates reduces the ductility of the beam. Plates can also be bonded to the sides of the beam to increase both the shear and flexural capacities as in Fig. I.B.1.2 (b). Plates can also be bonded to the compression face in a continuous beam by extending a tension face plate beyond the point of contraflexure. This is useful in inhibiting but not preventing debonding. Angle sections and channel sections can also be bonded to beams as in Fig. I.B.1.2 (c); they provide the characteristics of both tension face and side plates. Furthermore, any combination of these plating techniques can be used by the engineer.

Adhesive bonding gives a full interaction bond and hence gives the maximum increase in the beam's strength and stiffness. However an adhesive bond can fail in a brittle fashion by mechanical debonding, or through environmental factors. Hence, adhesively bonded plates are suitable for components with low shear forces, such as at serviceability loads or in slabs or in regions of beams with nominal stirrups, and require a friendly environment. Bolting gives a partial interaction bond, that is slip occurs between the plate and the beam, and hence bolted plates do not achieve the same increase in the beam's flexural strength and stiffness as adhesively bonded plates. However, bolted plates generally have a ductile failure. Bolting plates is particularly useful for increasing the shear capacity, as the ductile bolt shear connectors can accommodate the formation of cracks and the sliding action of shear cracks within the beam. Both adhesive bonding and bolting can provide the best combined benefits. It allows the maximum stiffness at serviceability with a guaranteed ductile failure mode at ultimate. Furthermore, it gives additional safety by providing two mechanisms to resist failure.

Terminating the plates at the points of contraflexure as shown in Fig. I.B.1.4 ensures that the plate ends do not debond due to flexural peeling, but shear peeling and axial peeling are still of concern.

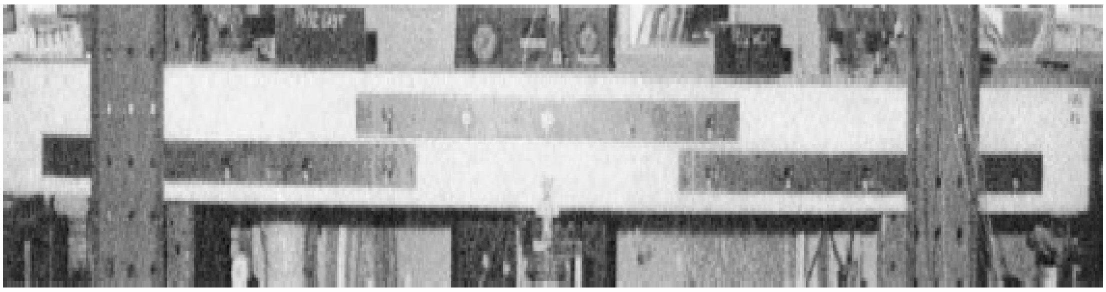

Fig. I.B.1.4 Plated continuous beam (Oehlers 2001).

Shear Strengthening

FRP systems have been shown to increase the shear strength of existing concrete beams and columns by wrapping or partially bonding FRP over the members (Malvar et al. 1995, Chajes et al. 1995, Norris et al. 1997, Kachlakev and McCurry 2000). Multiple options exist for shear strengthening including laminate bonding to the sides of the beam, U-jacketing around the bottom, and total wrapping of the beam as shown in Fig. I.B.1.5. Complete wrapping of the FRP system around the section on all four sides is the most efficient strengthening scheme and is most commonly used for column applications. In beam applications, the shear strength can be improved by wrapping the FRP system around three sides of the member (U-wrap) or bonding to the two sides of the member (ACI 440.2R-02). Although all three techniques have been shown to improve the shear strength of a member, complete wrapping of the section is the most efficient, followed by the three-sided U-wrap. Bonding to two sides of a beam is the least efficient scheme. The shear-resisting system can be in the form of continuous sheet or laminates with spacing. Fibers can be oriented either perpendicular to the axis of the beam or perpendicular to the potential shear cracks, or a combination of both orientations.

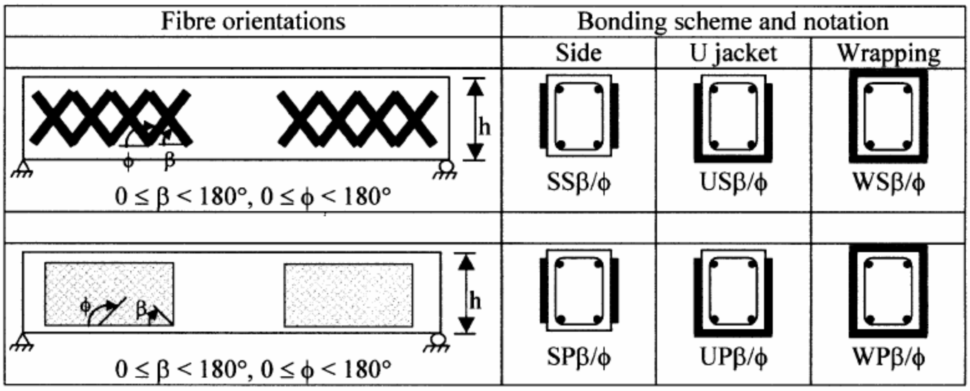

Fig. I.B.1.5 Shear strengthening schemes with FRP (Chen and Teng 2003).

Installation Procedure of FRP System

All systems are applied using similar processes. Figure I.B.1.6 shows the installation procedure by the wet lay-up technique (Nanni 1999). A series steps include:

(1) Surface preparation

(2) Applying a primer coat of resin to the surface with a roller and any small voids or bug holes are filled with putty

(3) Installing the actual FRP systems by sandwiching the dry fiber sheet between two layers of resin. The first resin or saturant coat is applied to the primed surface using a roller and the fiber sheet is gently pressed into the saturant. The system in this state is allowed to set several minutes. After approximately 30 minutes, a second coat of saturant is applied to complete the formation of the FRP material. If more than one ply of the fiber sheet is required to achieve the necessary strength, another coat of saturant must be applied followed by the fiber and a second coat of saturant

(4) Applying a final topcoat after the saturant has cured and is tack free (takes several hours and depends on environmental conditions). Various topcoats are available that provide protection from UV exposure, protection from abrasion, enhanced fire performance, or simply as an aesthetic coat to match the appearance of the original substrate

A method using prestressed FRP sheets for bonding to the concrete surface has been developed (Uomoto et al. 2002). The basic steps in the process are:

(1) Preparation of concrete surface in the usual manner (as for nonprestressed sheets)

(2) Prestressing the continuous FRP sheet

(3) Applying the adhesive resin along the plane of adhesion and bonding to the concrete surface

(4) Curing and hardening of hardener with the tensile force held in place. At the ends, the reinforcement is carried out using a sheet fitted with steel plates, which are anchored to the concrete using bolts

(5) Finally, removing the externally applied load once sufficient hardening has been achieved, introducing a compressive "prestress" in the FRP sheets. Excess sheets at the ends are cut off

The following steps were undertaken to strengthen a beam for increased shear capacity (Taljsten 2003):

(1) Design for higher loads

(2) Drawings and plans for strengthening

(3) Removal of weak concrete

(4) Casting of new concrete to the cleaned and dust free beam

(5) Hardening of concrete for 5 days

(6) Roughen the surface and drilling holes for anchorage

(7) Application of primer

(8) Application of adhesive and fiber wet out in three layers of fabrics

(9) Quartz sand in the last adhesive layer

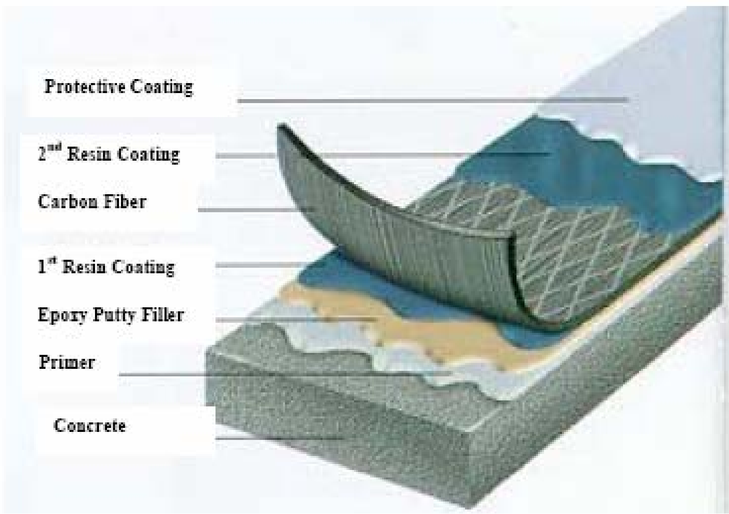

Fig. I.B.1.6 Installation procedure of FRP sheet system (Nanni 1999).

Anchorage

At the structure level, debonding is often found to initiate from the laminate ends where abrupt discontinuity arises. High concentration of shear and peel stress can be found in the bond line at such locations where shear cracks in concrete are likely to develop. Various methods of anchorages such as anchor bolts, L-shaped plates, and U-anchors, have since been devised with mixed success to prevent debonding from happening. Figure I.B.1.7 illustrates some of these schemes.

For shear strengthening of concrete members, the ends of FRP strips shall be adequately anchored to develop the design forces in the strips (Bridge Manual 2003). In a situation where a slab overlies a beam being strengthened (as with T-beams), the preferred approach is for intermittent slots to be cut in the slab and the FRP strips passed through the slab and anchored on the slab top surfaces.

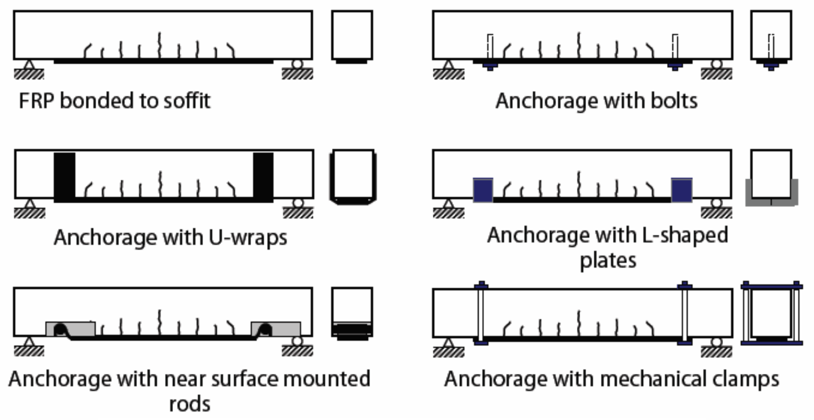

Fig.I.B.1.7 Anchorage schemes at laminate ends (Spadea et al. 1998).

Inspection and Verification (Mirmiran A et al. 2004; Nanni 1999)

Strengthening with FRP is highly dependent on the correct choice of materials and appropriate application of the composite. All stages of the strengthening process should be carefully supervised. On completion, a full inspection should be undertaken to determine the ‘as installed’ condition, which will act as a reference for subsequent inspections. Critical items for inspection include received materials, substrate repair, surface preparation, fiber orientation, debonding, curing of resin, adhesion, and cured thickness. Records of daily inspections may include conditions of the environment (e.g., temperature, humidity, and rain); surface conditions; surface profile; width of cracks not injected with epoxy; batch numbers; mixture ratios; mixing times; qualitative descriptions of the appearance of all mixed resins, primers, putties, saturants, adhesives, and coatings; observations of progress of cure of resins; conformance with installation procedures; adhesion test results (i.e., bond strength, failure mode, and location); FRP properties from tests of field sample panels or witness panels, if required; location and size of any delaminations or air voids; and general progress of work.

Once an FRP system is installed, verifying the quality of the installation is typically accomplished using two techniques. Hammer sounding or tap testing is used to find delaminations between the FRP and the substrate. Most systems permit a limited number of small delaminations without compromising their strength. Larger delaminations should either be injected with resin or repaired. The pull-off test is used to verify the bond to the substrate and the level of saturation in some systems. For some strengthening projects a final quality control step is performed by implementing a load test. After completing the FRP installation the structure is loaded and monitored to verify that the FRP strengthened structure is behaving per design.

I-B.3 Effectiveness, life-span and durability in relation to concrete deterioration

<u>In terms of enhancing structural performance</u>, fiber-wrap technology has been shown to be quite effective and will continue to be the number one application in construction (followed perhaps by FRP decks and FRP rebar), as indicated, for example, by about 80% of publications on this technology in the ASCE Journal of Composites for Construction. The structural effectiveness of FRPs in the rehabilitation of existing structural systems has repeatedly been demonstrated with full or large-scale structural tests. Experimental studies (Saadatmanesh and Ehsani 1991, Meier and Kaiser 1991, Ross et al. 1994, Sharif et al. 1994, Gold et al. 2000, Khalita and Nanni 2000, Balendran et al. 2001) have shown that reinforced concrete beams strengthened with externally bonded FRP laminates can exhibit ultimate load capacities as high as three times their

original capacity depending on the steel ratio, concrete strength, FRP ratio, FRP mechanical properties, properties of the bonding agent, and pre-existing level of damage of the beams. Flexural capacity can be increased by bonding composite plates, even though the chances of premature failure occurring due to cover delamination are high. For flexural strengthening, continuous unidirectional FRP composite plates, 1–2 mm thick, could provide strengthening equal to 6-mm-thick steel plates. Typical increases in strength for an 18.0-m-long beam vary from 23% for a 1-mm-thick FRP plate to over 69% for a 3-mm-thick FRP plate (Lane et al. 1997). The more layers of FRP are used, the higher flexural strength and shear capacity are achieved.

Prestressed concrete takes greater advantage of the high strength and strain capacity of the FRP materials and represents a more efficient use of the fiber-wrap technology (Burgoyne 1997). Benefits of prestressing FRP laminates include:

- Serviceability can be further improved and deflections reduced
- Effective at reducing crack widths and delaying the onset of cracking
- Relieve internal steel reinforcement strains
- Induced compressive stress resist fatigue failure
- Internal steel rebars yield at a higher proportion of the ultimate load
- More efficient use of the concrete and the FRP
- Opposes stresses due to both dead and live loads
- Reduce the risk of premature failure
- Ultimate capacity can be further increased
- Replace prestress that has been lost
- Shear capacity is increased by the longitudinal stresses induced

FRP materials have been attractive for strengthening of reinforced concrete beams. A large loss in beam ductility, however, occurs when they are used for flexural strengthening of reinforced concrete beams. A new pseudo-ductile FRP strengthening fabric has been developed at the Structural Testing Center at Lawrence Technological University (Grace et al. 2004). The fabric is unique in that it exhibits a yield plateau similar to that exhibited by steel in tension. The beams strengthened with the new fabric exhibited 24 to 42% higher ductility index than those strengthened with the carbon fiber sheets. This fabric was manufactured by triaxially braiding bundles of carbon and glass fibers in three different directions (+45, 0, and -45 degrees). These fibers were selected with different ultimate strains (0.35, 0.8, and 2.10 %) and were mixed in a way allowing them to fail successively generating a yield plateau.

<u>In terms of life-span,</u> however, there is not sufficient information to predict long-term performance with certainty, but predictions can be made in terms of current knowledge. In order to predict the long term behavior from the results of accelerated tests, statistical or analytical models can be employed. There are various models proposed in the literature, such as Arrhenius model, Eyring model, two-stage Temperature/Voltage models, and three-stage Temperature/Voltage and Humidity models. The Arrhenius model has been successfully applied to predict the strength retention of glass fiber-reinforced cement and glass fiber strand by Litherland et al. (1981). After that, this model also has been used in accelerated agency tests of FRP materials by Chin et al. (1998), Porter (1999), Dejke (2001), Gentry et al. (2002) and others.

As explained in Task I-A, <u>the greatest issue of concern is durability</u>, particularly the interface bond which is significantly affected by the deteriorated condition of the concrete substrate. Even though not enough data exist to verify long-term properties of the bond interfaces, since these materials have been used for only about ten years in the building industry, the experience from older steel plate bonding projects shows that many of these structures are still in use with no visible deterioration of the bond layer. If the right type of material is used, and if the strengthening work is carried out carefully, 30 years of use can be guaranteed (Taljsten 2004).

The performance of structures rehabilitated with FRPs in harsh and corrosive environments is a particular topic of concern (Neale 2000). Unacceptable reductions in mechanical properties can occur if resins with inadequate *moisture absorption* characteristics are employed. The performance of FRP rehabilitation methods in cold regions is also an important issue of concern. Tests on carbon FRP sheets subjected to natural and accelerated exposure have shown that these materials have adequate weather-proofing properties with regard to tensile strength and bond to concrete, as well as being quite durable to freezing and thawing. Some early studies indicated that *Freeze-thaw* cycling does not induce significant deterioration of the bond (Green et al. 1998), and that FRP strengthened concrete beams are not damaged significantly by exposure to *wet-dry* environments (Toutanji and Gomez 1997). However, recent accelerated tests for FRP-concrete specimens under opening-mode fracture, have shown decreases of about 50% in fracture toughness for freeze-thaw and wet-dry cycling (Davalos et al. 2004).

Most of the research into durability-related aspects of FRP strengthening and repair has only very recently been undertaken. As a result, there remain several unresolved questions and outstanding issues. Among these is the correlation of accelerated tests to actual field conditions, as well as the validity of extrapolating results from small-scale specimens to full-scale structures. Furthermore, the simulated laboratory environments vary considerably from one research establishment to another, with the result that it is extremely difficult to fully synthesize the existing test data. As such, it is virtually impossible at present to arrive at definite conclusions regarding the durability of a given FRP retrofit scheme in a particular environment. Efforts in standardizing accelerated laboratory test conditions would contribute greatly to basic understanding of the durability characteristics of FRP strengthening technologies.

I-B.1.4 Cost-effectiveness of the technology

Bridge repairing and strengthening using FRP materials is generally less costly than replacement and significantly shortens downtime for rehabilitation, which reduces inconvenience to the traveling public and economic loss to areas served. Strengthening a T-beam bridge in Rensselaer County, New York, using bonded FRP laminates demonstrated the application of innovative FRP materials, while showcasing cost-effective rehabilitation of a deteriorated reinforced concrete bridge to improve its capacity and extend its service life (Hag-Elsafi et al. 2001 and 2004). Total cost of the rehabilitation project was estimated at 25% of the cost required for complete structural replacement.

However, when evaluating first-costs in relation to traditional methods, FRP technology will probably be more expensive. The key advantages of FRPs are often overlooked in relation to high materials and manufacturing costs. However, several developments have changed this situation over the past few years (Vistasp et al. 2000, Van Den Einde et al. 2003): (1) techniques such as pultrusion, resin transfer molding, filament winding and semi-automated manufacturing of large components have led to advances in low cost FRP manufacturing; (2) downturn in defense spending and the resulting need for new markets has spurred renewed efforts in reducing the costs of both raw materials and manufacturing processes; and (3) designs of these new materials in conjunction with conventional structural materials rather than individual component replacement or complete advanced composite designs, have shown that technical efficiency can be achieved within competitive economical constraints.

Although FRPs are generally more expensive than conventional construction materials, repairing and retrofitting using FRP patching and wrapping instead of traditional methods can nevertheless be very cost-effective in the long run if their use leads to fewer re-repairs (Sen 2003). A direct comparison of the unit price basis may not be appropriate. When installation is included in the cost comparison, FRPs can compete with conventional materials. The low weight of FRP reduces transportation, labor and operational expenses and allows some prefabrication to take place at the factory, which reduces time at the job site. The use of FRP materials significantly shortens downtime for rehabilitation, which reduces inconvenience to the traveling public and economic loss to the area served, especially for highly-populated metropolitan areas. If the comparisons include lower overall life cycle costs, and the attendant relaxation of the crippling need for large maintenance budgets, FRP can have a significant advantage.

I-B.1.5 Limitations for application of the technology

The use of FRP composites is now a widely-accepted solution for the repairing and strengthening of reinforced concrete structures. FRP repairs and strengthening schemes offer many well documented benefits for the rehabilitation of many existing concrete buildings and bridges. It must, however, be mentioned that composites do suffer from some disadvantages, primary among them being: (a) higher initial materials cost, (b) lack of familiarity in most areas (outside aerospace related application areas), and (c) lack of comprehensive standards and design guidelines at present (Karbhari and Zhao 2000). Indeed, in some cases, the lack of understanding or design guidance is a major limitation to wider implementation of FRP materials in concrete applications. Some concerns still remain about FRP's long-term creep characteristics, and aging due to ultraviolet rays or degradation of bond forces with time. Also, a full understanding of failure behavior and design models that would reflect the improvements in strength and stiffness of rehabilitated or retrofitted concrete structural members still need to be developed.

Applications where existing FRP systems may not be useful include correcting punching shear problems in slabs or footings, correcting vibration problems, and providing greater compression strength to walls (Nanni 1999). In cases where FRP is useful, it should be recognized that there are reasonable limits to the additional strength afforded with FRP. Typically, increases in strength up to 50% are reasonable. It is also important to recognize that in cases where FRP is being used to address a deterioration problem, the FRP system serves as an effective barrier for the ingress of chlorides or alkali, but it is unable to prevent on-going corrosion caused by deleterious materials already trapped inside, and may conceal visual signs of deterioration (Sen 2003, Nanni 1999). The source of the deterioration should always be addressed and corrected prior to installing FRP. A common example is corrosion of steel reinforcement in a concrete beam or column. FRP should never be used to contain corrosion. FRP will not stop corrosion from progressing, and, in the case of externally bonded FRP systems, the corrosion will eventually result in failure due to debonding.

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Sub-Task I-B.2: Analysis from Surveys

In this section, we synthesize the survey information and summarize the results in tabulated “fact-sheet” form for easy interpretation and use. The individual responses to the survey are given in Appendix B.

Table I.B.2.1 summarizes the responses on the applications of surface bonded FRP technology. On average, almost 5 projects were carried out by each state/agency. It appears that most information sources that were used for determining the applicability of the technology came from published information/available literature and soliciting advice from research/university collaborators, materials/technology suppliers, or contractors familiar with technology implementation. Of the specifications available for the application of the technology, most states/agencies (respondents thereafter) have materials selection and/or qualification procedures, construction specifications and/or procedures, and quality control and/or quality assurance procedures. Only 4 out of 14 respondents have performance assessment procedures. Of the application situations of the technology, beams rank highest (50%) in all situations, followed by slabs and pier caps (29% each), arches and columns (14% each), and piers (7%). For the purpose of the application, repair and/or strengthening rank highest, fewer for corrosion mitigation and seismic upgrade. Quite surprisingly, 60% overall pre-repair conditions for the repaired structures are severe deterioration. Of the specific existing conditions where the technology was applied, loss of concrete section appears in almost all projects (8 out of 9); damage due to salt exposure ranks second (5 out of 9); then corrosion of rebar (4 out of 9); damage due to alkali-silica exposure (3 out of 9). Other conditions include broken strands, broken tendons by collision damage, and freeze-thaw cracking.

Overall cost for application of the technology is satisfactory (11 out of 12) both in relation to expected service-life and in relation to other repair methods. Furthermore, the overall satisfaction is very high (84%, or 11 out of 13) for the technology; and 12 out of 13 respondents would recommend the technology for other concrete repair/retrofit projects. For cost-effective implementation of the technology, the ideal conditions would be minor concrete deterioration, a need of strengthening instead of replacing, and a need to meet public demands and avoid load-posting. There are several limitations for the effective application of this technology. For examples, DOTs are quite concerned about the use of FRP technology even though it is a proven technology; there is no comprehensive set of design specifications; the technology is limited if significant portion of core concrete is lost, or members are too deteriorated; the technology would not be used if there is no redundancy, or steel were ruptured. In addition, uncontrolled environment, field conditions for mixing materials, need for access, sharp corners, inspection, and maintenance also limit the application of this technology.

Table I.B.2.2 summarizes the implementation protocols for surface bonded FRP technology. Usually, there are several entities involved in a project. In the survey, almost all projects (92%) had a materials and technology supplier involved; 83% of projects had in-house design/materials division and applications contractor involved; 58% had research institute/university involved; and 17% had consulting engineering company involved. Of the responsibilities of a research institute/university involved, specification of materials and lay-up architecture and details, instrumentation during or after application, and testing and evaluation after application rank highest (71% each); followed by supervision of field work, and QA and QC application measures (57% each); advisory to the DOT, selection of technology supplier, and inspection after application (43% each); coordination among all entities involved, studies for service life-span and cost effectiveness, and studies for long-term performance and durability rank the lowest. It appears that all material/technology suppliers have responsibilities of materials specification and supply, materials test data and physical properties/design values, and application methods and field work. Training or certification of installation personnel ranks second. It seems that material/technology suppliers tend not to offer field inspection and assessment of needed repair/retrofit, instrumentation during or after application, or studies for performance and/or durability. Since there are only three responses to the responsibilities of a consulting company, the results are not clear. Similarly, there are only four responses to the question 5. It seems, however, a field application sub-contractor, different from the materials and technology supplier, has responsibility for all field work (4 out of 4). Of the degree of satisfaction with project management, availability of qualified research/university consultant ranks highest (very satisfied), followed by successful application of the technology and availability of qualified materials and technology supplier (satisfied to very satisfied). Successful QA/QC or evaluation method during applications ranks lowest (indifferent). All others rank between indifferent to satisfied. There are several preferred or suggested management approaches for the effective implementation of the technology. Nestore Galati (UMR) suggested a research/design method and obtained most successful results when the designer interacted with the contractor in order to avoid misinterpretations of drawings/specifications. Scott Newbauer (IA DOT) suggested central office lead and to use state forces.

Table I.B.2.3 summarizes evaluations and assessments of surface bonded FRP technology. It appears that the most likely evaluations of surface bonded FRP repair procedures conducted for field projects were done for pre-repair conditions, to determine applicability of the technology, field testing and correlations with predictions, and qualification in-house (33% each). There is no evaluation or assessment for proprietary methods available from vendor, critical QA/QC issues for field application, design methods and performance limits, inspection methods after construction, or cost/benefit studies. There is no response to the second question – evaluations for acceptable pre-repair substrate conditions. There is only one response to the third question – evaluations for acceptable repair procedures during applications. Evaluations were performed on crack injection for selecting materials/techniques, surface treatment of substrate concrete, forming and patching to restore section, surface preparation of patched and pre-existing concrete for application of resin/fiber system, and finish coat application for UV/fire/other protective coating. Of evaluations conducted immediately after completion of repair applications, both respondents conducted evaluations on inspection for overall quality of finished repair, inspection for delamination or “missed” spots by sounding/tapping or other technique, tests on either lab-cured or field-cured coupon samples for strength/integrity of interface, tests on actual repaired sections for strength and integrity of interface, and full scale proof or load test on structure or components. For the follow-up or long-term evaluations, all four respondents performed visual inspections, photographs and documentation; two of them did load tests; and one conducted measurements. Of the degree of satisfaction with evaluations and assessment procedures, both respondents were satisfied or very satisfied on procedures for pre-repair conditions, procedures immediately after work completion, and procedures for long-term conditions; but both were indifferent to procedures during repair work applications. Osman Hag-Elsafi (NY DOT) suggested load testing as an approach for evaluation/assessment of the effective implementation of the technology.

Table I.B.2.1 Application of Surface Bonded FRP Technology

Category	**Results (%) (applied responses / total respondents)**
1. State/agency experience	100% (14/14), average about 5 projects per agency
2. Information sources	
• By soliciting advice	86% (12/14)
• Based on previous experience	14% (2/14)
• Based on recommendations of DOT or owner	21% (3/14)
• Based on published information	64% (9/14)
• Other-developed guidelines/principles, seminar	21% (3/14)
3. Specifications available	
• Material specifications	86% (12/14)
• Construction specifications	64% (9/14)
• Quality control procedures	71% (10/14)
• Performance assessment procedures	29% (4/14)
4. Application situations	
• Pier caps	29% (4/14)
• Piers	7% (1/14)
• Beams	50% (7/14)
• Slabs	29% (4/14)
• Walls	0% (0/14)
• Arches	14% (2/14)
• Columns	14% (2/14)
• Other	0% (0/14)
5. Purpose	
• Repair	36% (5/14)
• Strengthen	36% (5/14)
• Both repair and strengthen	36% (5/14)
• Corrosion mitigation/repair	21% (3/14)
• Seismic upgrade	14% (2/14)
• Other-proof of concept	7% (1/14)
6. Overall pre-repair conditions	
• No apparent deterioration	20% (2/10)
• Mild deterioration	20% (2/10)
• Moderate deterioration	30% (3/10)
• Severe deterioration	60% (6/10)
• Other	0% (0/10)
7. Specific existing conditions	
• Loss of concrete section	89% (8/9)
• Corrosion of rebar	44% (4/9)
• Damage due to salt exposure	56% (5/9)
• Damage due to alkali-silica exposure	33% (3/9)
• Other- broken strand & tendon, cracking, aged	56% (5/9)
8. Overall cost assessment	
• Satisfactory to expected service-life	92% (11/12)
• Satisfactory to other repair methods	92% (11/12)

9. Overall satisfaction • Very satisfied • Satisfied • Indifferent • Dissatisfied • Very dissatisfied • Other Recommend the technology for other projects	 46% (6/13) 38% (5/13) 15% (2/13) 0% (0/13) 0% (0/13) 0% (0/13) 92% (12/13)
10. Conditions for the implementation and suggestions • Minor concrete deterioration, otherwise the repair become expensive (Nestore Galati, UMR) • Economy, where there is a need to strengthen rather than to replace (Joe Lenzini & Tom Domagalski, IL DOT) • Cost effective if the bridge does not meet current demands and may require load-posting (Rick Pilcher, MO DOT) • Concern: Leave some gaps at the bottom to allow moisture to escape (Mike Mohseni, CO DOT) • Pre-loaded bridge to put girder into tension, then patch (Matt Farrar, ID DOT)	
11. Limitations for the effective application • DOTs are very concerned about the use of FRP technology even though it is a proven technology (Nestore Galati, UMR) • Problem because there is no comprehensive set of design specifications (Rich Carter, CalTrans) • Uncontrolled environment; field conditions for mixing materials; need to be able to access; sharp corners a problem; inspection; maintenance (Osman Hag-Elsafi, NY DOT) • Would not use if there was no redundancy. Would not use if steel were ruptured (Jim Gutierrez, CalTrans) • Limited if significant portion of core concrete is lost (Scott Neubauer, IA DOT) • Limited if members are too deteriorated (Joe Lenzini & Tom Domagalski, IL DOT) • Too costly; there are more cost effective methods (Daniel Hsiao, UT DOT)	

Table I.B.2.2 Implementation Protocols for Surface Bonded FRP Technology

Category	Results (%) (applied responses / total respondents)
1. Entities involved	
• Research institute/university	58% (7/12)
• Materials and technology supplier	92% (11/12)
• Consulting engineering company	17% (2/12)
• In-house design/materials division	83% (10/12)
• Contractor different from materials supplier	83% (10/12)
• Other-DOT assessment unit, FHWA observed	8% (1/12)
2. Responsibilities of research Institute/university	
• Advisor to DOT	43% (3/7)
• Coordination among all entities involved	14% (1/7)
• Specification of materials, and lay-up	71% (5/7)
• Selection of technology supplier	43% (3/7)
• Selection of applications contractor	29% (2/7)
• Supervision of field work	57% (4/7)
• QA and QC application measures	57% (4/7)
• Instrumentation during or after application	71% (5/7)
• Inspection after application	43% (3/7)
• Testing and evaluation after application	71% (5/7)
• Studies for service life-span & cost effective	14% (1/7)
• Studies for long-term performance & durability	29% (2/7)
3. Responsibilities of material/technology supplier	
• Field inspection & assessment of needed repair	14% (1/7)
• Materials specification or supply	100% (7/7)
• Materials test data & design values	100% (7/7)
• Application methods or field work	100% (7/7)
• Selection of field application sub-contractor	29% (2/7)
• Training/certification of installation personnel	57% (4/7)
• Responsibility for all field work	29% (2/7)
• QA and QC application measures	29% (2/7)
• Instrumentation during or after application	14% (1/7)
• Inspection after application	29% (2/7)
• Testing and evaluation after application	29% (2/7)
• Studies for performance and/or durability	14% (1/7)
4. Responsibilities of consulting company	(only three respondents)
• Advisory to the DOT	0% (0/3)
• Coordination among all entities involved	0% (0/3)
• Specification of materials or design of lay-up	33% (1/3)
• Design details for target performance	0% (0/3)
• Prepare contract documents	0% (0/3)
• Administer contract/advertise bid	0% (0/3)
• Selection of technology supplier	0% (0/3)
• Selection of application contractor	0% (0/3)
• Supervision of field work	0% (0/3)

• QA and QC application measures • Instrumentation during or after application • Inspection after application • Testing and evaluation after application • Studies for service life-span & cost effective • Studies for long-term performance & durability • Other-prepare plans & construction documents	0% (0/3) 0% (0/3) 0% (0/3) 0% (0/3) 0% (0/3) 0% (0/3) 67% (2/3)
5. Responsibilities of a field application sub-contractor, different from materials supplier • Field inspection & assessment of needed repair • Responsibility for all field work • Training/certification of installation personnel • QA and QC application measures • Instrumentation during or after application • Inspection after application • Testing and evaluation after application • Studies for performance or durability • Other-traffic control	(only four respondents) 0% (0/4) 100% (4/4) 25% (1/4) 25% (1/4) 0% (0/4) 25% (1/4) 25% (1/4) 0% (0/4) 25% (1/4)
6. Satisfaction with project management by following scale: 5-very satisfied; 4-satisfied; 3-indifferent; 2-dissatisfied; 1-very dissatisfied: • Coordination of all entities involved • Availability of qualified research/university consultant • Availability of qualified materials and technology supplier • Availability of qualified field applications contractor/personnel • Availability of qualified consulting company • Availability of qualified in-house design/material personnel • Successful application of the technology • Successful QA/QC or evaluation methods during applications • Successful testing and assessment methods of the applications • Successful evaluations of the technology thereafter	Average scale of 7 respondents: 3.93 5.00 4.14 3.50 3.50 3.40 4.29 3.00 3.80 3.67

7. Preferred/suggested management approach for the effective implementation of the technology

- A research/design method obtained most successful results when the designer interacted with the contractor in order to avoid misinterpretations of drawings/specifications (Nestore Galati, UMR)
- Mix of procedures & performance specifications; performance specifications on materials; procedures specifications on application. Would prefer a performance specifications, ultimately (Rick Carter, CalTrans)
- Central office lead; no set process; use state forces (Scott Newbauer, IA DOT)

Table I.B.2.3 Evaluations and Assessments of Surface Bonded FRP Technology

Category	**Results (%) (applied responses / total respondents)**
1. Evaluations of surface bonded FRP repair procedures for field project(s)	
• Pre-repair conditions to determine applicability	33% (3/9)
• Pre-approvals for matl selection/qualifications	11% (1/9)
• Proprietary methods available from vendors	0% (0/9)
• Critical QA/QC issues for field implementation	0% (0/9)
• Design methods and performance limits	0% (0/9)
• Inspection methods after construction	0% (0/9)
• Field testing and correlations with predictions	33% (3/9)
• Cost/benefit studies	0% (0/9)
• Other-qualification in-house	33% (3/9)
2. Evaluations/inspections conducted for determining acceptance pre-repair substrate condition	N/A
3. Evaluations/inspections conducted for determining acceptable repair procedures during applications:	(only one respondent)
• Crack injection for selecting matls/tech	Yes
• Surface treatment of substrate concrete	Yes
• Forming and patching to restore section	Yes
• Surface preparation of patched and pre-existing concrete for application of resin/fiber sys.	Yes
• Resin/fiber applications and finishing	No
• Finish coat application for UV/fire/other	Yes
• Other	No
4. Evaluations/assessments conducted immediately after completion of repair applications	(only two respondents)
• Inspection for overall quality of finished repair	100% (2/2)
• Inspection for delamination or “missed” spots by sounding/tapping or other technique	100% (2/2)
• Tests on either lab-cured or field-cured coupon samples for strength/integrity of interface	100% (2/2)
• Tests on actual repaired sections for strength and integrity of interface	100% (2/2)
• Full scale proof or load test on structure/comp.	100% (2/2)
• Tests on either lab-cured or field-cured coupon samples for effectiveness/performance of coat	0% (0/2)
• Other	0% (0/2)
5. Follow-up or long-term evaluations/assessments	(only four respondents)
• Visual inspections, photographs and document.	100% (4/4)
• Measurements	33% (1/4)
• Load tests	50% (2/4)
• Other	0% (0/4)

6. Degree of satisfaction with evaluations and assessment procedures, by scale: 5-very satisfied; 4-satisfied; 3-indifferent; 2-dissatisfied; 1-very dissatisfied • Procedures for pre-repair conditions • Procedures during repair work applications • Procedures immediately after work completion • Procedures for long-term conditions • Other	Average scale of 2 respondents 4.50 3.00 4.50 4.00 N/A
7. Preferred or suggested evaluation/assessment approach for the effective implementation • Load testing (Osman Hag-Elsafi, NY DOT) • Long term – investigate the bond coating (Scott Neubauer, IA DOT)	

Sub-Task I-B.3: Combined Synthesis

The analyses from the literature and surveys in Sub-Task I-B.1 and Sub-Task I-B.2, respectively, are used in this section to provide a combined synthesis of findings, with the purpose of answering the questions of concern by PennDOT.

What are applicable situations for use of the FRP technology?

FRP composites used in new bridge construction and rehabilitation have provided bridge engineers with innovative solutions for today's infrastructure problems. The most promising applications of FRP composites in civil engineering are in structural rehabilitation. Recent FRP rehabilitation projects have had a broad range of varied characteristics, and have been carried out in regions encompassing a wide variety of environmental conditions. The reported laboratory and field test results are quite encouraging not only for pristine FRP-concrete specimens, but also for pre-cracked members, and samples subjected to some form of environmental exposure. The applicable situations of FRP composites can be summarized as:

- Beams
- Slabs and decks
- Columns
- Piers and pier caps
- Walls
- Arches
- Silos
- Tanks
- Pipes
- Piles
- Other corroded bridge elements

FRP plate bonding technique is now popular in the in-situ enhancing of concrete beams in bridges, particularly in structures that have either partially lost their embedded flexural steel through corrosion or that need upgrading due to increased load requirements. Many bridges to which the technique was applied are single span and simply supported. From the survey results gathered in this project, beams rank highest (50%) for all the application situations, followed by slabs and pier caps (29% each), arches and columns

(14% each), and piers (7%). Most information sources that were used for determining the applicability of the technology came from published information/available literature and soliciting advice. Most responding states had materials selection, construction, and quality control procedures. Repair and/or strengthening were the main purpose of the application. About 60% of overall pre-repair conditions for the repaired structures were characterized by severe deterioration and loss of concrete section. The overall satisfaction was very high with this technology; and almost all states recommended the technology for other concrete repair/retrofit projects.

How is industry or states implementing FRP technology for concrete repair?

The technical and organizational approaches that industry and states have adopted to implement this technology are synthesized as follows.

Technical approaches

Substrate repair and surface preparation: The behavior of concrete members strengthened or retrofitted with FRP systems is highly dependent on a sound concrete substrate, and also proper preparation and profiling of the concrete surface. The issues for substrate repair and surface preparation are discussed in ACI 440.2R-02 and NCHRP Report 514. Specific guidelines for a particular FRP system should be obtained from the FRP system manufacturer. *Substrate repair* includes removal of defective concrete, repair of defective reinforcement, restoration of concrete cross section, and injection of cracks. *The surface of substrate* must be cleaned of all laitance, dust, dirt, oil, curing compound, and existing coatings. It must also be free of moisture and frost before installing the FRP system. Surface irregularities should be ground smooth within acceptable tolerances. Bug holes and voids should be filled with epoxy putty. The minimum radius of sharp corners should be 1/2 in. (13 mm). For contact-critical application, surfaces to be wrapped should be flat or convex.

FRP repair systems: There are three common FRP system forms suitable for the strengthening of structural members. These are wet layup systems, prepreg systems, and precured systems. Other FRP systems include near-surface mounted, FRP rigid rod and flexible strand or cable. Wet lay-up and precured FRP systems may be prestressed to improve their performance. The prestrain in carbon FRP systems should be limited to 50% of the ultimate strain.

Application methods: Composites can be applied in three ways: adhesive bonding, wet lay-up, and resin infusion. Adhesive bonding and wet lay-up are the most widely used.

Plating techniques: Currently, there are tension face plates to improve the flexural capacity, side plates to improve the shear capacity, and combination plates to improve both shear and flexural capacities. These plates can be bolted or adhesively bonded. Recently, the fastened method has been developed, where the strengthening strip is entirely mechanically attached to the concrete surface using multiple, small, and distributed powder-actuated fasteners without any bonding. The fastened method can provide increased ductility over the bonded method. Terminating the plates at the points of contra-flexure ensures that the plate ends do not disbond due to flexural peeling, but shear peeling and axial peeling are still of concern.

Shear strengthening: Options for shear strengthening include bonding to the sides of the beam, U-jacketing around the bottom, and total wrapping of the beam. Complete wrapping of the section is the most efficient method to improve the shear strength, followed by U-wrap. Bonding to two sides of a beam is the least efficient scheme.

Anchorage: Anchor bolts, L-shaped plates, and U-anchors, have been devised with mixed success to prevent debonding from happening. In a situation where a slab overlies a beam being strengthened (as with a T-beam), the preferred approach is for intermittent slots to be cut in the slab and the FRP strips passed through the slab and anchored on the slab top surfaces.

Installation procedures: All systems are applied using similar processes. A series of installation steps by the *wet lay-up* technique include: surface preparation ⟶ applying primer coat of resin and putty ⟶ installing FRP systems by sandwiching the dry fiber sheet between two layers of resin ⟶ curing ⟶ applying final topcoat. The basic steps using *prestressed FRP sheets* for bonding are: surface preparation ⟶ prestressing FRP sheet ⟶ applying adhesive resin ⟶ bonding FRP sheets ⟶ curing and hardening with tensile force ⟶ removing applied load.

Inspection and verification: The main issues for the construction inspection including responsibility and criteria for the inspector, methods of inspection, records keeping, critical items requiring inspection, sampling frequency and location, and acceptance criteria are recommended in NCHRP Report 514.

Organizational approaches

Entities involved: There are several entities involved in a project. From our survey, most projects (over 83%) had materials and technology supplier, in-house design/materials division, and applications contractor involved; over 50% of the projects had research institute/university involved; and only a few projects had consulting engineering company involved.

Responsibilities of research institute/university: The major responsibilities of a research institute/university include: (1) specification of materials and lay-up architecture and details, (2) instrumentation during or after application, and (3) testing and evaluation after application. Supervision of field work, and QA and QC application measures are the second. Advisor to DOT, selection of supplier, and inspection after application are the third.

Responsibilities of material/technology supplier: All suppliers had responsibilities of (1) materials specification and supply, (2) materials test data and design values, and (3) application methods and field work. The suppliers tended not to offer field inspection and assessment of needed repair/retrofit, instrumentation during or after application, or studies for performance and/or durability.

<u>*Responsibilities of consulting company and sub-contractor*</u>: The responsibilities of consulting company and sub-contractor are not clear since there were not enough responses in the survey. However, it seems that sub-contractors had responsibility for all field work.

Generally, the availability of qualified research/university consultant, materials supplier, and successful application of the technology were ranked as “very satisfied”. The most likely evaluations conducted for field projects were pre-repair conditions, field testing, and in-house qualifications. From the survey, there was no evaluation for available proprietary methods, critical QA/QC issues, design methods, inspection methods after construction, or cost/benefit studies. It appears that evaluations conducted immediately after repair applications included overall quality of finished repair, delamination by sounding/tapping or other technique, tests on either lab-cured or field-cured coupon samples for strength/integrity of interface, tests on actual repaired sections for strength and integrity of interface, and load test on structure. Visual inspections, photographs and documentation were follow-up or long-term evaluations conducted in all projects. Load test was another major follow-up assessment tool used for the effective evaluation of the technology.

What are the effectiveness, life-span and durability of the various applications of this technology when concrete has been deteriorated by salts, alkali-silica reactions, etc?

Reinforced concrete beams strengthened with externally bonded FRP laminates can exhibit ultimate load capacities as high as three times their original capacity. For flexural strengthening, continuous unidirectional FRP composite plates, 1–2 mm thick, could provide strengthening equal to 6-mm-thick steel plates. The more layers of FRP are used, the higher flexural strength and shear capacity are achieved. Prestressing FRP laminates can provide many benefits and represent a more efficient use of the technology. A significant loss in beam ductility can occur when FRP materials are used for flexural strengthening. A new triaxially braided ductile fabric can provide 24 to 42% higher ductility index than commonly used carbon fiber sheets.

There is not sufficient information to predict long-term performance with certainty, but predictions can be made from the results of accelerated tests using different statistical or analytical models such as Arrhenius model. There is not enough data to verify long-term properties of the bond interfaces; however, if the right type of material is used, and if the strengthening work is carried out carefully, 30 years of use can be guaranteed. Unacceptable reductions in mechanical properties can occur if resins with inadequate moisture absorption characteristics are employed. CFRP sheets have adequate weather-proofing properties with regard to tensile strength and bond to concrete, as well as being quite durable to freezing and thawing. Freeze-thaw and wet-dry cycling will induce significant deterioration of the bond. There remain several unresolved questions and outstanding issues on durability-related aspects of FRP strengthening and repair. It is virtually impossible at present to arrive at definite conclusions regarding the durability of a given FRP retrofit scheme in a particular environment.

What is the cost effectiveness of the technology?

Currently, *first-costs* of FRP technology will be more expensive in relation to traditional methods due to high materials and manufacturing costs. However, when installation is included in the cost comparison, FRPs can compete with conventional materials. If the comparisons include reduced overall *life cycle costs*, and the attendant relaxation of the crippling need for large maintenance budgets, FRP can have a significant advantage. From our survey, almost all states/agencies (11 out of 12) were satisfied with the overall cost of the technology in relation to both expected service-life and other repair methods.

Are there limitations of the fiber wrap technology as it relates to the type of repair/rehabilitation application proposed under this research project?

The FRP technology has disadvantages and limitations which include:

- Higher initial materials cost
- Lack of familiarity in most areas (outside aerospace related application areas)
- Lack of comprehensive standards and design guidelines at present
- Long-term durability is still not completely defined
- A full understanding of failure behavior and design models still needs to be developed

- Increases in strength may not be achieved beyond 50%
- Unable to prevent on-going corrosion and may conceal visual signs of deterioration
- Limited effectiveness for correcting punching shear problems in slabs or footings
- Limited effectiveness for correcting vibration problems
- Limited effectiveness for providing greater compression strength to walls
- Limited effectiveness if significant portion of core concrete is lost
- Limited effectiveness if members are too deteriorated
- Some engineers feel that FRP would not be used if there was not enough redundancy in the system. Others are skeptical of using FRP if steel strands or bars are ruptured, or have potential of rupturing.

In addition, uncontrolled environment, field conditions for mixing materials, need for adequate access, sharp corners, inspection, and maintenance also limit the applications of this technology.

Section 3 - Task I-C: Defining Ideal Candidate Bridge for Repair

The above tasks in combination with the findings in this section permitted us to define an "ideal" candidate bridge, in close collaboration with District-3 Engineers. This section for Task I.C – Defining Ideal Candidate Bridge for Repair, includes Classification of District-3 Concrete T-beam Bridges (Sub-Task I.C.1), Field Visitation and Inspection (Sub-Task I.C.2), and Recommendations for Selecting Candidate Bridges (Sub-Task I.C.3).

Sub-Task I-C.1: Classification of District-3 Concrete T-beam Bridges

This sub-task discusses the basis for classification of District-3 PennDOT bridges for suitability of repair with surface-bonded composites. The information is presented in two major sections: (1) criteria, and (2) illustrative explanations.

I-C.1.1 Criteria for Classification of T-beam Bridges

For the purpose of having criteria for visiting and inspecting a representative range of concrete bridges in the inventory provided, the structures are classified based on a weighted average of the following four characteristics:

1. Age
2. Span Length
3. ADT/ADTT
4. Photographic Indication of Damage

For each of these characteristics, scores are assigned based on favorability for repair. Each score (generally 1-10) is assigned in an attempt to collect and organize similar bridges into groups for the purpose of classification into three classes, with the potential for repair assigned as: prime (class #1), moderate (class #2), and low (class #3).

This section discusses the four characteristics listed above, the weighted percentages assigned to each, and the classification of bridges into three classes.

I-C.1.1.1 Age of Bridge

For the purpose of classification, the ages of the bridges are assigned a rating of 1 through 10, where a bridge with a rating of 10 is most suitable for repair and a bridge with a rating of 1 is least favorable for repair. Older bridges are rated lower in this category because of the possible shorter remaining service-life. Also, due to global deterioration of the bridge, replacement rather than repair may be the most economical alternative. Based on age, the following scores are assigned for each age category:

Age Category	Score	Bridges in Category
1955-1973	10	12
1950-1954	9	8
1944-1949	8	3
1942-1945	7	8
1940-1941	6	14
1935-1939	5	21
1930-1934	4	29
1925-1929	3	21
1920-1924	2	9
Older than 1920	1	3
	Total =	128

I-C.1.1.2 Span Length

For the purpose of classification, the span lengths of the bridges are assigned a rating of 1 through 10, where a bridge with a rating of 10 is most suitable for repair and a bridge with a rating of 1 is least favorable for repair. Because of the similarity in beam size and spacing, a longer span will experience more critical loads than a shorter span. Hence, a longer span is more favorable for repair because of strength concerns. Also, longer bridges are inherently more costly for replacement. The repair cost effectiveness is assumed to be more favorable for longer bridges.

Span (ft)	Score	Bridges in Category
80+	10	5
60-79	9	5
50-59	8	7
45-49	7	9
40-44	6	10
35-39	5	16
30-34	4	24
25-29	3	33
20-24	2	15
Less than 20	1	4
	Total =	128

I-C.1.1.3 ADT and ADTT

For the purpose of classification, the ADT and ADTT (average daily traffic and average daily truck traffic respectively) of the bridges are assigned a rating of 1 through 10, where a bridge with a rating of 10 is most suitable for repair and a bridge with a rating of 1 is least favorable for repair. A bridge which is carrying more volume of traffic is more favorable for repair than a bridge that does not carry much traffic at all. Because of limited resources, it is more sensible to repair bridges that are used more frequently. Also, with the increase of truck traffic, the probability of carrying future critical loads increases.

ADT	**Score**	**Bridges in Category**
10,000 +	10	6
5,000-9,999	9	14
3,000-4,999	8	19
2,500-2,999	7	9
1,500-2,499	6	14
1,000-1,499	5	13
500-999	4	11
300-499	3	17
150-299	2	11
Less than 150	1	14
	Total =	128

ADTT	**Score**	**Bridges in Category**
1,000 +	10	3
500-999	9	7
400-499	8	7
300-399	7	11
200-299	6	8
100-199	5	18
75-99	4	14
50-74	3	15
25-49	2	13
Less than 25	1	32
	Total =	128

I-C.1.1.4 Photographical Evidence of Damage

For the purpose of classification, damages indicated by photographical information provided by PennDOT are scored into the following rating categories: 3, 2, 1, and 0.

Based on NCHRP Report 514 pages I-12 thru I-13 [1] and a journal article by Kutarba et al. (2004) [2], bridges were evaluated based on the similarity in damage compared to those damages described in the literature.

Bridges which received a rating of 3 are possibly prime candidates for repair. Bridges which received a rating of 2 may be candidates for repair, however field inspection is strongly suggested to determine if the damage to these bridges is suitable for repair. Bridges with a rating of 1 are either not likely candidates for repair based on photographs, or the photographs may be unclear in regard to the damage. Bridges with a rating of 0 are, judging by photographs, either not requiring any repairs or not applicable for repairs.

The reason for ratings of 3, 2, 1 and 0 is for numerical scoring of the bridges. Each bridge with a rating of 3 received a score of 10, while each bridge with a rating of 2 received a score of 6.67. Each bridge with a rating of 1 received a score of 3.33, and finally, all bridges with a rating of 0 received a score of 0.

Rating	**Score**	**Bridges in Category**
3	10	30
2	6.67	36
1	3.33	17
0	0	45
	Total =	128

I-C.1.1.5 Weighted Values for Classification

In order to aid in the selection of bridges for further evaluation, the scores assigned for each characteristic are tabulated and averaged based on weighted averages.

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[1] NCHRP Report 514 (2004), "Bonded Repair and Retrofit of Concrete Structures Using FRP Composites." Transportation Research Board of the National Academies.

[2] Kutarba, M.P., Brown J.R., and Hamilton H.R. (2004). "Repair of Corrosion Damaged Concrete Beams with Carbon Fiber-Reinforced Polymer Composites." COMPOSITES 2004 Convention and Trade Show.

The age of the bridge is considered because of both economical and repair concerns. Older bridges are more likely to reach sooner their design life. Also, older bridges will generally have concrete of lesser quality (due to deterioration) which will be important in the implementation of the FRP technology.

The span length of the bridges is an important parameter. Because most of these bridges in the inventory are of similar designs, longer spans will equate to more critical load conditions. Hence, it will likely be more economical to apply the technology to bridges with longer span lengths.

An important factor in the classification of bridges is the ADT and ADTT. Larger traffic volumes will result in increased probability of critical loading, and it is more cost effective to repair bridges which are used more often.

Finally, in order to determine if a bridge is a good candidate for visitation and repair, bridges are scored based on the type and extent of damage indicated by the photographs provided by PennDOT District-3.

Each characteristic is weighted as follows:

1. Age (15%)
2. Span Length (10%)
3. ADT/ADTT (35%) (17.5% & 17.5%)
4. Photographic Indication of Damage (40%)

I-C.1.1.6 Tentative Groups for Visitation

Once a total score is calculated, the bridges are classified into three tentative groups for visitation. The details for the classification of bridges are provided in Table I.C.1.1.

Class 1: Prime Candidate for Repair (Score of 70-100%). This set of bridges is likely a prime candidate for repair by FRP technology. Field investigation is suggested for the following purpose: On some of these bridges, the damage may be so severe that the most economical course of action may be to replace these structures. An ideal candidate bridge will likely be chosen from this set of bridges.

Class 2: Moderate Candidate for Repair (Score of 50-69%). These bridges are likely candidates for repair. However, field investigation of these bridges is suggested to closely examine the type and extent of damage, as well as the cost-benefit of applying the FRP technology to these bridges.

Class 3: Low Candidate for Repair (Score of 0-49%). Because of age, size, level of traffic, damage type or lack of damage, these bridges are not the prime focus for the FRP technology. Some of these bridges may benefit from the FRP repair technology, but they may not be the most economical choice for this type of repair.

The breakdown of bridges is as follows:

Rating	**Score**	**Bridges in Category**
Class 1	70-100%	19
Class 2	50-69%	45
Class 3	0-49%	64
	Total =	128

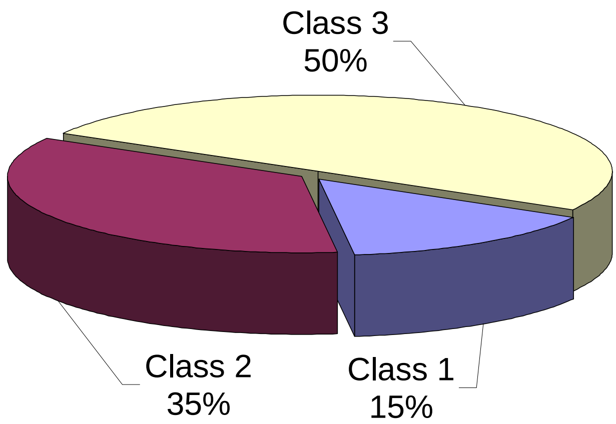


Fig.I.C.1.1. Graphical representation of the number of bridges in each class.

Table I.C.1.1. Initial Classification of Bridges in District 3-0

Class 1 Prime Candidate for Repair: (Total = 19)

Bridge	Rank	Span	Span Score	Yr Built	Yr Score	ADT	ADT Score	ADTT	ADTT Score	Visual	Visual Score
59-0045-0310-2011	84.8	30	4	38	5	10300	10	650	9	3	10.00
08-0014-0580-0000	80.8	27	3	57	10	2500	7	250	6	3	10.00
41-0118-0020-1109	79.8	31	4	30	4	5600	9	410	8	3	10.00
41-0654-0210-1810	79.3	48	7	34	4	6900	9	200	6	3	10.00
54-0522-0210-0000	79.3	39	5	29	3	6200	9	400	8	3	10.00
49-0225-0180-0000	78.0	66	9	27	3	3000	8	200	6	3	10.00
41-0220-0131-1268	77.7	45	7	41	6	11000	10	1100	10	2	6.67
41-0442-0160-0000	77.3	97	10	27	3	3800	8	130	5	3	10.00
59-0304-0060-1859	77.0	45	7	41	6	3011	8	90	4	3	10.00
58-4002-0020-0000	74.3	33	4	35	5	3700	8	120	5	3	10.00
59-0015-0091-1230	73.9	50	8	30	4	9900	9	1300	10	2	6.67
49-0405-0260-0942	73.5	27	3	30	4	5700	9	110	5	3	10.00
19-1014-0052-0442	73.3	29	3	38	5	3300	8	100	5	3	10.00
19-4006-0082-0000	70.5	48	7	34	4	1000	5	150	5	3	10.00
41-2005-0052-0000	70.5	39	5	25	3	2400	6	210	6	3	10.00
58-0249-0062-0000	70.5	48	7	32	4	1500	6	90	4	3	10.00
59-0045-0430-1068	70.4	29	3	37	5	10300	10	650	9	2	6.67
08-4034-0140-1580	70.3	50	8	52	9	350	3	35	2	3	10.00
19-0442-0050-0264	70.3	37	5	31	4	2100	6	100	5	3	10.00

Class 2 Moderate Candidate for Repair: (Total = 45)

Bridge	Rank	Span	Span Score	Yr Built	Yr Score	ADT	ADT Score	ADTT	ADTT Score	Visual	Visual Score
19-2009-0010-0594	69.5	41	6	30	4	2500	7	70	3	3	10.00
58-4024-0110-0000	68.8	46	7	34	4	650	4	110	5	3	10.00
41-0118-0290-0000	67.7	41	6	43	7	2600	7	300	7	2	6.67
41-0118-0030-1331	67.4	39	5	30	4	5600	9	410	8	2	6.67
08-4013-0250-1496	66.8	25	3	27	3	1600	6	160	5	3	10.00
49-0061-0016-1066	66.2	62	9	34	4	5400	9	160	5	2	6.67
41-0014-0220-0000	65.4	27	3	30	4	3200	8	600	9	2	6.67
54-0522-0090-1932	65.4	55	8	29	3	4200	8	300	7	2	6.67
49-3010-0110-0000	65.3	48	7	31	4	636	4	60	3	3	10.00
41-0014-0260-0368	64.9	34	4	28	3	3200	8	600	9	2	6.67
19-1025-0050-0000	64.4	41	6	41	6	3700	8	110	5	2	6.67
41-2014-0380-0000	64.2	23	2	36	5	6900	9	340	7	2	6.67
19-4036-0012-0000	63.8	48	7	26	3	500	4	60	3	3	10.00
49-4018-0010-0649	63.5	27	3	22	2	3400	8	40	2	3	10.00
54-0522-0230-1430	62.9	24	2	27	3	6300	9	400	8	2	6.67
59-1001-0110-2058	62.8	32	4	21	2	1400	5	80	4	3	10.00
49-1039-0020-0000	62.3	30	4	30	4	500	4	50	3	3	10.00
56-0220-0550-0000	61.4	30	4	25	3	3400	8	340	7	2	6.67
58-0249-0112-2535	59.7	23	2	52	9	1500	6	90	4	2	6.67

Class 2 (continued)

Bridge	Rank	Span	Span Score	Yr Built	Yr Score	ADT	ADT Score	ADTT	ADTT Score	Visual	Visual Score
19-0042-0690-1821	59.3	29	3	55	10	5400	9	350	7	1	3.33
49-4012-0250-1032	58.9	48	7	34	4	2200	6	100	5	2	6.67
59-0045-0350-0000	58.6	27	3	40	6	10300	10	650	9	1	3.33
41-0118-0230-0644	58.3	80	10	43	7	2600	7	300	7	1	3.33
58-0249-0322-0000	57.9	51	8	36	5	1260	5	80	4	2	6.67
41-0042-0050-1036	57.4	41	6	40	6	1700	6	62	3	2	6.67
19-0042-0090-0000	57.2	33	4	40	6	2200	6	75	4	2	6.67
19-1025-0050-1308	57.2	33	4	41	6	2900	7	70	3	2	6.67
59-2001-0020-0000	57.0	33	4	32	4	440	3	20	1	3	10.00
41-2001-0130-0652	56.4	27	3	37	5	2600	7	80	4	2	6.67
19-4045-0030-1814	56.3	35	5	34	4	220	2	20	1	3	10.00
08-0414-0200-0000	56.2	30	4	26	3	1500	6	200	6	2	6.67
59-3006-0050-0282	56.0	23	2	45	7	120	1	12	1	3	10.00
19-4003-0010-0082	55.4	174	10	23	2	1200	5	80	4	2	6.67
08-4031-0200-0000	55.3	23	2	29	3	360	3	35	2	3	10.00
58-2014-0152-0000	55.0	34	4	37	5	100	1	7	1	3	10.00
49-0225-0380-0000	54.9	29	3	34	4	1500	6	130	5	2	6.67
49-3018-0120-1195	54.9	35	5	38	5	1000	5	90	4	2	6.67
19-0093-0110-1085	52.3	27	3	73	10	3900	8	75	4	1	3.33
49-0054-0090-0835	52.3	25	3	59	10	3000	8	90	4	1	3.33
54-0104-0230-0000	52.1	27	3	30	4	5600	9	450	8	1	3.33
54-0522-0110-0469	52.1	55	8	29	3	4200	8	300	7	1	3.33
49-0642-0130-0000	51.4	29	3	30	4	2200	6	70	3	2	6.67
49-2002-0010-1145	50.7	44	6	35	5	740	4	40	2	2	6.67
58-0015-0820-2041	50.5	37	5	42	7	12300	10	2100	10	0	0.00
54-2013-0010-0000	50.3	23	2	22	2	200	2	20	1	3	10.00

Class 3 Low Candidate for Repair: (Total = 45)

Bridge	Rank	Span	Span Score	Yr Built	Yr Score	ADT	ADT Score	ADTT	ADTT Score	Visual	Visual Score
19-0239-0060-0511	49.2	33	4	26	3	1800	6	40	2	2	6.67
54-0522-0040-0000	49.1	35	5	27	3	3000	8	360	7	1	3.33
56-0220-0530-0988	48.1	33	4	25	3	4300	8	340	7	1	3.33
41-1005-0130-0000	47.7	39	5	40	6	300	3	4	1	2	6.67
41-0414-0080-0000	45.9	25	3	37	5	300	3	30	2	2	6.67
54-4012-0020-0000	45.9	25	3	38	5	460	3	40	2	2	6.67
54-2004-0030-1543	44.9	33	4	41	6	160	2	13	1	2	6.67
59-0045-0440-1302	44.8	33	4	38	5	10300	10	650	9	0	0.00
58-2014-0092-0000	44.2	50	8	34	4	100	1	7	1	2	6.67
41-2019-0080-0000	40.7	27	3	37	5	140	1	10	1	2	6.67
54-2010-0040-1803	40.7	25	3	37	5	120	1	10	1	2	6.67
41-0014-0100-0000	40.3	43	6	28	3	4000	8	600	9	0	0.00
54-0522-0150-1440	40.3	40	6	28	3	5200	9	400	8	0	0.00
19-3008-0100-0039	39.9	23	2	33	4	270	2	20	1	2	6.67

Class 3 (continued)

Bridge	Rank	Span	Span Score	Yr Built	Yr Score	ADT	ADT Score	ADTT	ADTT Score	Visual	Visual Score
54-0522-0140-0000	39.8	67	9	29	3	4200	8	300	7	0	0.00
56-0154-0050-0000	39.3	50	8	35	5	360	3	70	3	1	3.33
41-2083-0040-1285	39.2	26	3	34	4	140	1	10	1	2	6.67
08-0414-0230-1008	38.8	26	3	19	1	1500	6	200	6	1	3.33
54-0104-0120-2462	38.8	29	3	30	4	5200	9	450	8	0	0.00
58-0549-0020-0000	38.5	32	4	53	9	2700	7	160	5	0	0.00
54-4012-0130-0000	38.1	35	5	39	5	560	4	50	3	1	3.33
58-0006-0610-0000	37.8	19	1	42	7	6500	9	260	6	0	0.00
58-0414-0350-0000	37.5	31	4	41	6	2500	7	300	7	0	0.00
54-0235-0150-0000	33.5	94	10	30	4	1100	5	100	5	0	0.00
49-4001-0090-0066	33.0	25	3	41	6	2700	7	160	5	0	0.00
54-2003-0020-0000	32.6	23	2	49	8	150	2	6	1	1	3.33
54-0235-0140-0239	32.5	78	9	30	4	1100	5	100	5	0	0.00
58-2014-0040-0946	32.3	38	5	59	10	1200	5	40	2	0	0.00
58-2014-0050-0815	32.3	38	5	59	10	1200	5	40	2	0	0.00
54-4003-0030-0254	32.0	44	6	48	8	1100	5	50	3	0	0.00
58-0414-0430-0190	31.5	42	6	57	10	330	3	50	3	0	0.00
49-3010-0160-0263	31.3	84	10	40	6	750	4	70	3	0	0.00
54-2016-0050-0000	31.0	29	3	42	7	1100	5	100	5	0	0.00
58-1002-0050-0000	30.1	27	3	17	1	600	4	60	3	1	3.33
54-0235-0120-0094	29.5	43	6	30	4	1100	5	100	5	0	0.00
58-0006-0020-0000	29.5	67	9	24	2	3500	8	32	2	0	0.00
59-2009-0040-0000	29.0	23	2	30	4	1500	6	200	6	0	0.00
58-0414-0110-0261	28.8	36	5	57	10	310	3	40	2	0	0.00
58-2005-0220-0320	28.5	33	4	42	7	950	4	80	4	0	0.00
59-3001-0010-1018	27.1	30	4	25	3	200	2	15	1	1	3.33
58-1026-0040-0304	27.0	35	5	59	10	300	3	15	1	0	0.00
58-3014-0120-0000	26.8	27	3	57	10	350	3	35	2	0	0.00
49-4006-0160-0589	25.0	27	3	60	10	480	3	5	1	0	0.00
58-0414-0540-1036	25.0	19	1	54	9	330	3	50	3	0	0.00
54-3012-0080-0000	24.5	27	3	37	5	520	4	80	4	0	0.00
58-2016-0260-0543	24.3	38	5	42	7	900	4	20	1	0	0.00
54-2004-0020-0000	23.6	23	2	22	2	160	2	13	1	1	3.33
49-2001-0020-0000	23.3	23	2	28	3	120	1	10	1	1	3.33
58-3006-0060-3546	22.8	33	4	50	9	150	2	15	1	0	0.00
54-2012-0090-0091	21.8	29	3	51	9	210	2	2	1	0	0.00
49-3011-0010-0238	21.5	27	3	57	10	70	1	6	1	0	0.00
56-0154-0150-1564	21.0	26	3	35	5	360	3	70	3	0	0.00
56-4008-0080-0000	21.0	30	4	50	9	120	1	10	1	0	0.00
49-3003-0020-0645	20.8	24	2	22	2	1000	5	90	4	0	0.00
58-1022-0090-0000	20.3	31	4	38	5	300	3	30	2	0	0.00
59-1003-0140-0921	19.5	35	5	36	5	385	3	10	1	0	0.00
56-4020-0010-0000	18.5	26	3	49	8	140	1	10	1	0	0.00
56-1010-0030-0980	18.0	19	1	51	9	110	1	10	1	0	0.00
56-1002-0110-0000	14.5	22	2	41	6	100	1	10	1	0	0.00
56-1002-0140-0000	13.5	15	1	40	6	100	1	10	1	0	0.00

Class 3 (continued)

Bridge	Rank	Span	Span Score	Yr Built	Yr Score	ADT	ADT Score	ADTT	ADTT Score	Visual	Visual Score
49-4020-0010-1014	12.3	31	4	21	2	230	2	20	1	0	0.00
49-2005-0040-1415	11.5	24	2	30	4	130	1	10	1	0	0.00
58-1004-0120-1913	11.5	27	3	19	1	300	3	9	1	0	0.00
49-4022-0010-1591	10.3	23	2	22	2	230	2	10	1	0	0.00

I-C.1.2 Illustrations of Overall and Visual Damage Classifications for District-3 PennDOT Bridges

This supplemental information is provided to better illustrate with examples the classification of concrete T-beam bridges. This section describes: (1) Overall classification of bridges; (2) Examples of damage and repair from existing literature; and (3) Visual examples illustrating levels of damage for four categories (high, moderate, low and none).

I-C.1.2.1 Overall Classification of Bridges

In order to illustrate the classification of the bridges based on visual damage and the effect of other categories on the overall ranking, several examples of each classification are provided in this section. Visual damage was subjectively ranked based on similarities for repair of sample bridges reported in NCHRP 514 (2004) [1], and a journal article by Kutarba et al. (2004) [2]. It was found that many of the bridges in District-3 experienced damage levels similar to the examples given in the above sources. The examples that follow compare photos provided by District-3 with the photos from the NCHRP report to show similarities (Examples I.C.1.1-I.C.1.7).

Each example shows the overall and specific rankings for each category. Also, there are notes on each example which indicate the effects of each specific ranking on the overall score. Table I.C.1.2 shows the scoring and ranking for the visual examples provided.

I-C.1.2.2 Examples of Damage and Repair

Visual classification of photographs was based on two reports: NCHRP Report 514 (2004) [1] and Kutarba et al. (2004) [2]. Figure I.C.1.2 shows an example of damage repaired by the FRP technology from reference [1]. Figure I.C.1.3 shows an example from reference [2].

Table I.C.1.2. Scoring and Ranking for Examples Provided.

Class 1											
Bridge	**Rank**	**Span**	**Span Score**	**Yr Built**	**Yr Score**	**ADT**	**ADT Score**	**ADTT**	**ADTT Score**	**Visual**	**Visual Score**
08-4034-0140-1580	70.3	50	8	52	9	350	3	35	2	3	10.00
59-0045-0310-2011	84.8	30	4	38	5	10300	10	650	9	3	10.00
Class 2											
Bridge	**Rank**	**Span**	**Span Score**	**Yr Built**	**Yr Score**	**ADT**	**ADT Score**	**ADTT**	**ADTT Score**	**Visual**	**Visual Score**
08-4013-0250-1496	66.8	25	3	27	3	1600	6	160	5	3	10.00
59-1001-0110-2058	62.8	32	4	21	2	1400	5	80	4	3	10.00
54-0522-0090-1932	65.4	55	8	29	3	4200	8	300	7	2	6.67
Class 3											
Bridge	**Rank**	**Span**	**Span Score**	**Yr Built**	**Yr Score**	**ADT**	**ADT Score**	**ADTT**	**ADTT Score**	**Visual**	**Visual Score**
19-3008-0100-0039	39.9	23	2	33	4	265	2	20	1	2	6.67
58-1002-0050-0000	30.1	27	3	17	1	600	4	60	3	1	3.33

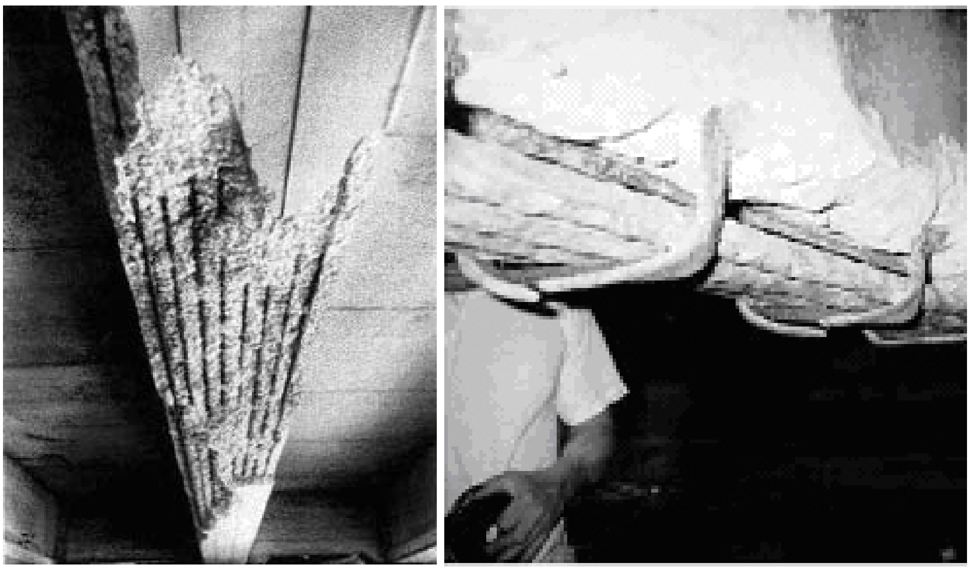


Fig. I.C.1.2. An example of damage that was repaired by the FRP technology [1].

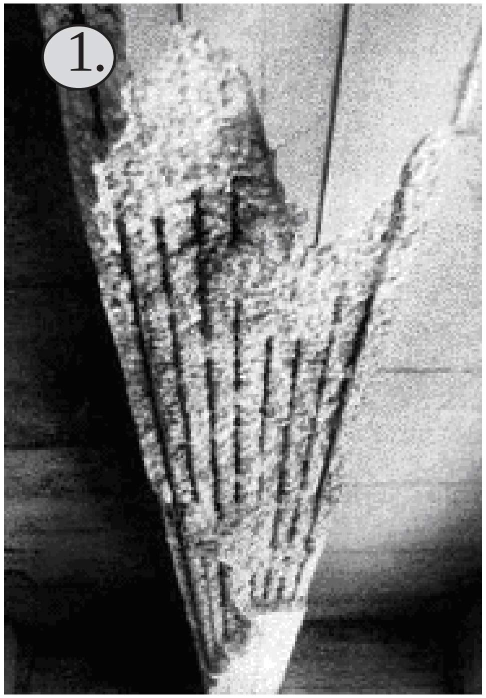
(a) (b)

Fig. I.C.1.3. An example of (a) damage and (b) repair by FRP technology [2].

The following seven examples illustrate overall scores and rankings of damage based on the photographs above in Figures I.C.1.2 and I.C.1.3.

Example 1.C.1.1: Bradford County #08-4034-0140-1580

Class #1: Overall Score (70.3/100)



Details:

Span(ft): 50
Ranking: 8/10

Year Built: 1952
Ranking: 9/10

ADT: 354
Ranking: 3/10

ADTT: 35
Ranking: 2/10

Visual Damage:
Ranking 10/10

Number of Spans: 1

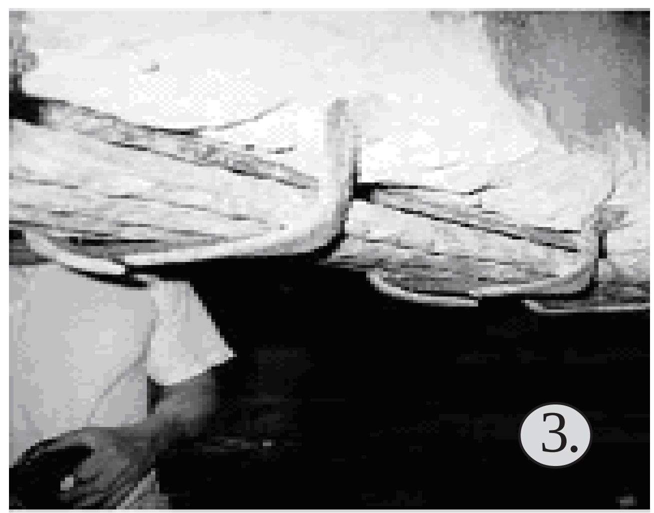
Status: Open

Road: SR 4034

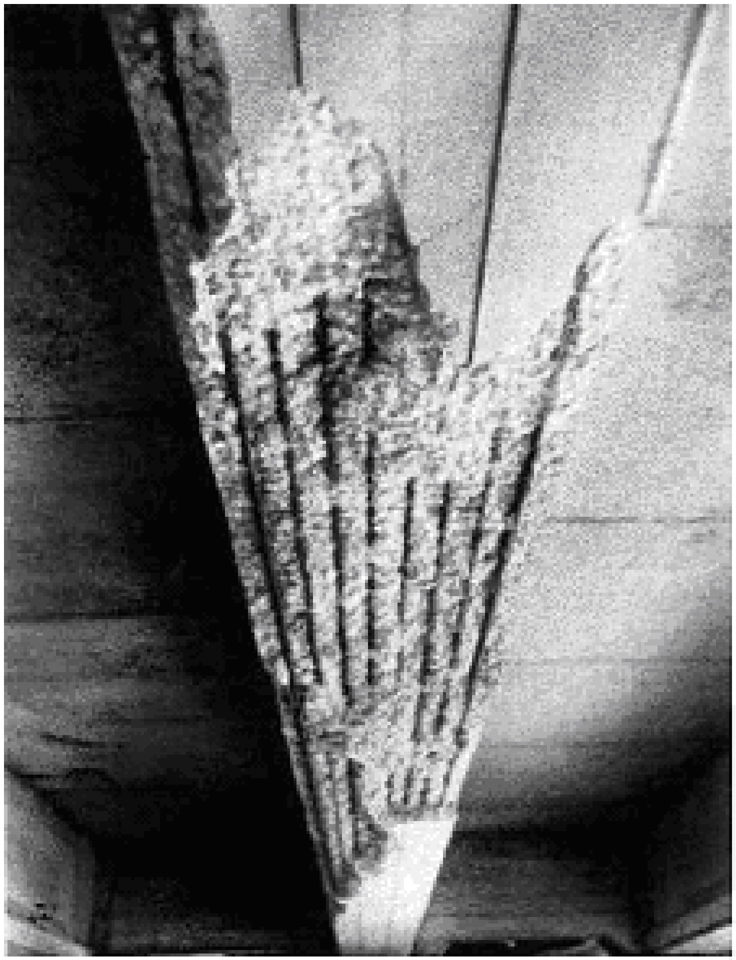
Photos 1 and 3: NCHRP Report 514, examples of a T-beam suitable for FRP repair techonology

Photo 2: Provided by PennDOT for bridge #08-4034-0140-1580

Note: This bridge carries low traffic volume. But the combination of span, age and visual damage resulted in a ranking of about 70 out of 100, or the lower end of the population of Class #1 bridges.



Example 1.C.1.2: Union County #59-0045-0310-2011
Class #1: Overall Score (84.8/100)



Details:

Span (ft): 30
Ranking: 4/10

Year Built: 1938
Ranking: 5/10

ADT: 10310
Ranking: 10/10

ADTT: 648
Ranking: 9/10

Visual Damage:
Ranking: 10/10

Number of Spans: 1

Status: Open

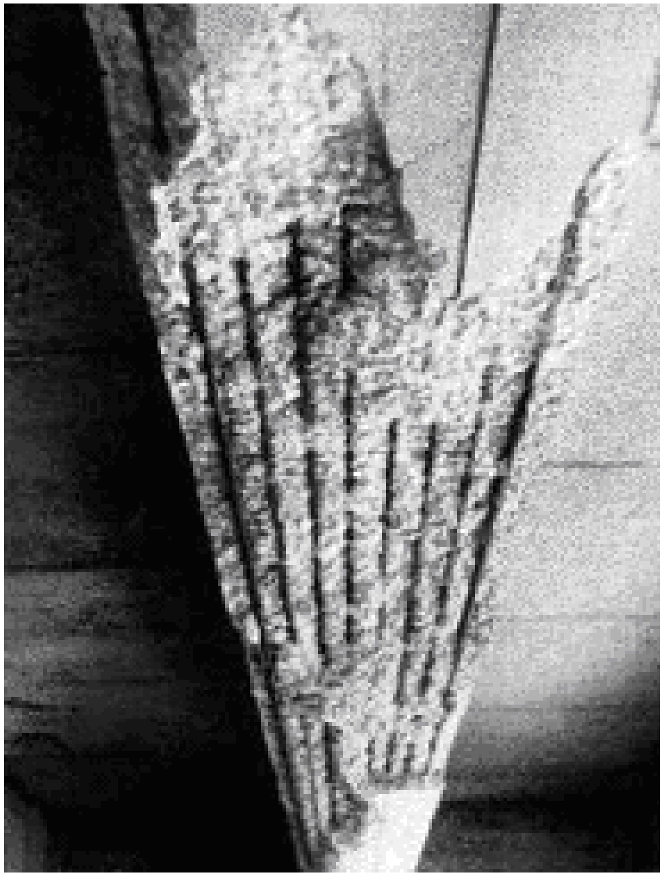
Road: SR 45

Photo on left: NCHRP Report 514, an example of a T-beam suitable for FRP repair techonology

Photo on right: Provided by PennDOT for bridge #59-0045-0310-2011

Note: This is a likely candidate for repair. This bridge scored high because of its large traffic volume and its similarities in damage to the NCHRP Report 514. This bridge ranked high even though it scored relatively low in the span and age category.

Example 1.C.1.3: Bradford County #08-4013-0250-1496
Class #2: Overall Score (66.8/100)



Details:

Span(ft): 25
Rank: 3/10

Year Built: 1927
Ranking: 3/10

ADT: 1606
Ranking: 6/10

ADTT: 161
Ranking: 5/10

Visual Damage:
Ranking: 10/10

Number of Spans: 1

Status: Open

Road: SR 4013

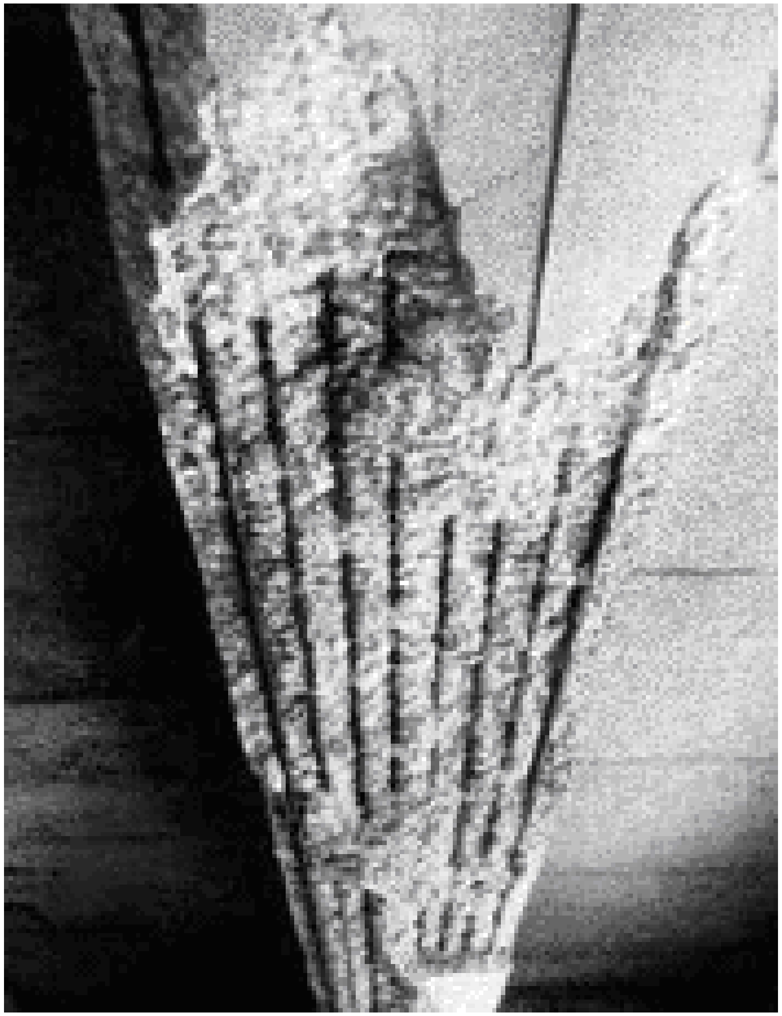
Photo on left: NCHRP Report 514, an example of a T-beam suitable for FRP repair techonology

Photo on right: Provided by PennDOT for bridge #08-4013-0250-1496

Note: Though this bridge scored low in the span and age category, this may be a good candidate bridge due to the traffic volume. Also, the bridge shows similar damage to that illustrated in the NCHRP Report 514. This bridge scores high in Class #2 (score 50-70).

Example 1.C.1.4: Union County #59-1001-0110-2058

Class #2: Overall Score (62.8/100)



Details:

Span (ft): 32
Ranking: 4/10

Year Built: 1921
Ranking: 2/10

ADT: 1379
Ranking: 5/10

ADTT: 81
Ranking: 4/10

Visual Damage:
Ranking: 10/10

Number of Spans: 1

Status: Open

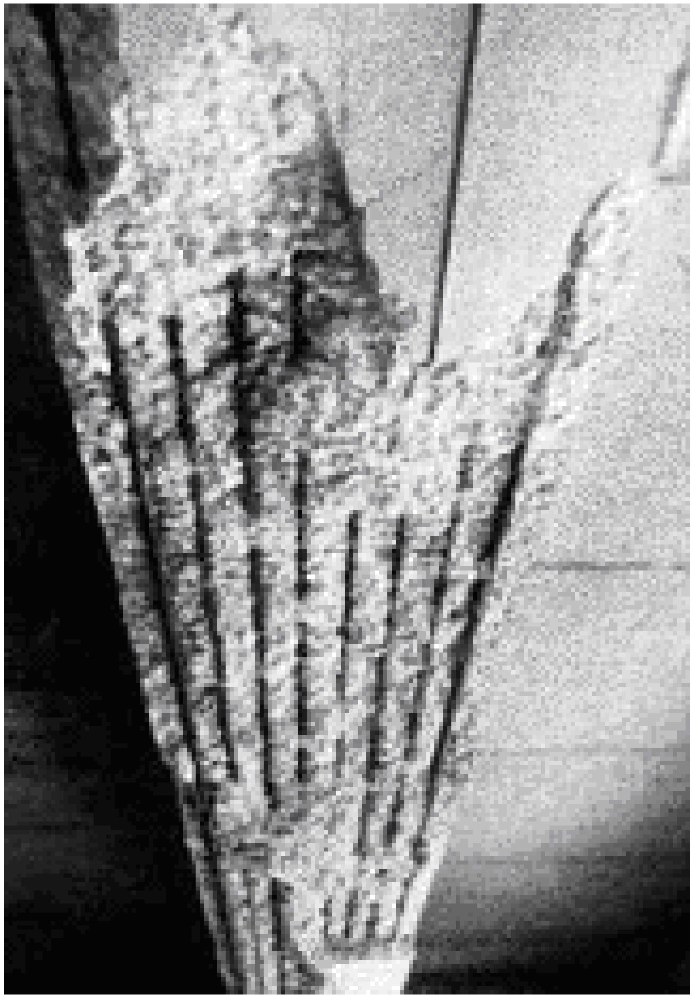
Road: SR 1001

Photo on left: NCHRP Report 514, an example of a T-beam suitable for FRP repair techonology

Photo on right: Provided by PennDOT for bridge #59-1001-0110-2058

Note: This bridge scores moderately in all of the categories except the visual category. In the visual category, the bridge scores very high, because the damage shown is very similar to that described in the NCHRP Report 514. This bridge scores in the middle of the Class #3 range (score 50-70).

Example 1.C.1.5: Snyder County #54-0522-0090-1932
Class #2: Overall Score (65.4/100)



Details:

Span (ft): 55
Ranking: 8/10

Year Built: 1929
Ranking: 3/10

ADT: 4195
Ranking: 8/10

ADTT: 299
Ranking: 7/10

Visual Damage:
Ranking: 6.67/10

Number of Spans: 2

Status: Open

Road: SR 522

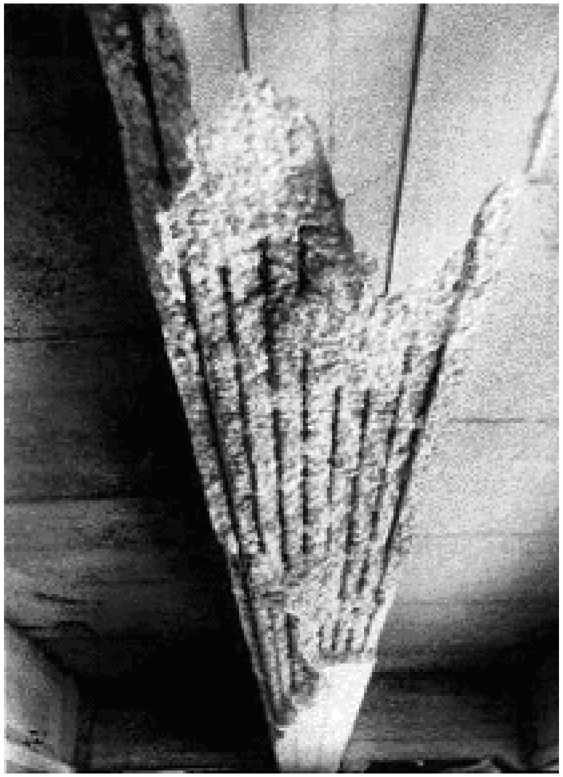
Photo on left: NCHRP Report 514, an example of a T-beam suitable for FRP repair techonology.

Photo on right: Provided by PennDOT for bridge #54-0522-0090-1932

Note: Though this bridge carries a lot of traffic, and ranks high in the span category, it is ranked into Class #2 because of the extent of the visual damage. This bridge would be suitable for repair by FRP technololgy.

Example 1.C.1.6 Columbia County #19-3008-0100-0039

Class #3: Overall Score (39.9/100)



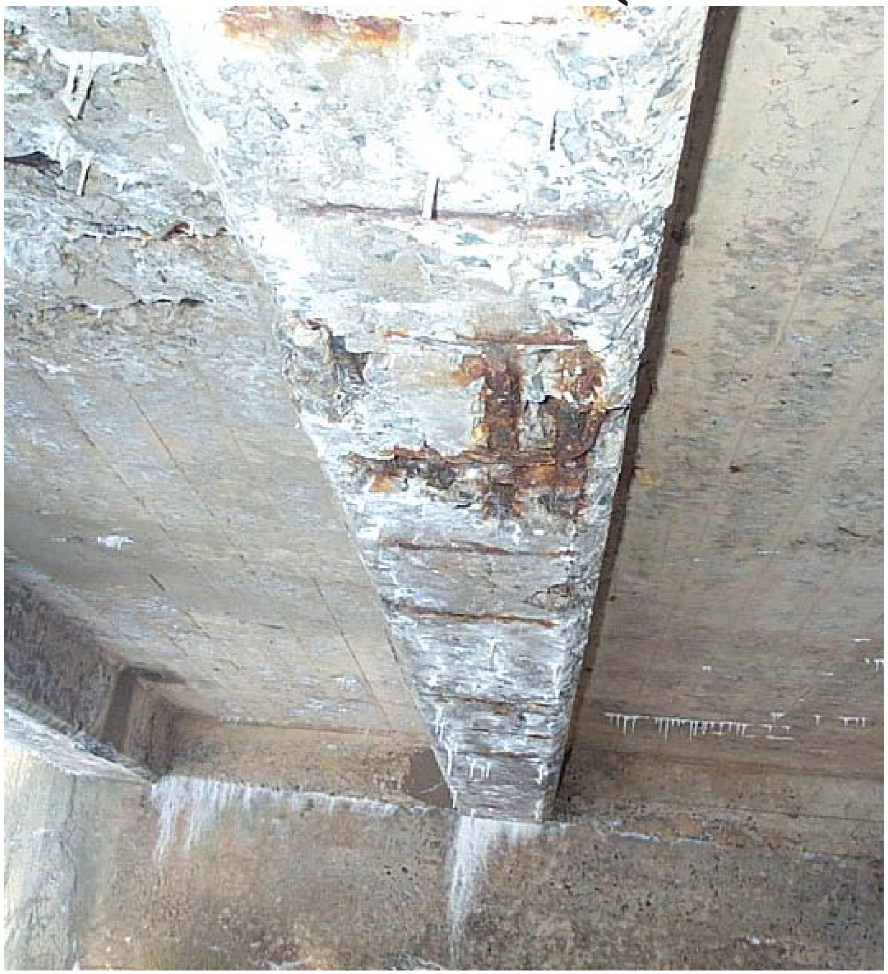


Photo on left: NCHRP Report 514, an example of a T-beam suitable for FRP repair techonology.

Photo on right: Provided by PennDOT for bridge #19-3008-0100-0039.

Note: This bridge ranked low overall because of low traffic volume, short span, and age. Also, the visual damage was not as severe as many of the other bridges. This is an example of a bridge which may not be a good candidate for the FRP technology.

Details:

Span(ft): 23
Ranking: 2/10

Year Built: 1933
Ranking: 4/10

ADT: 265
Ranking: 2/10

ADTT: 20
Ranking: 1/10

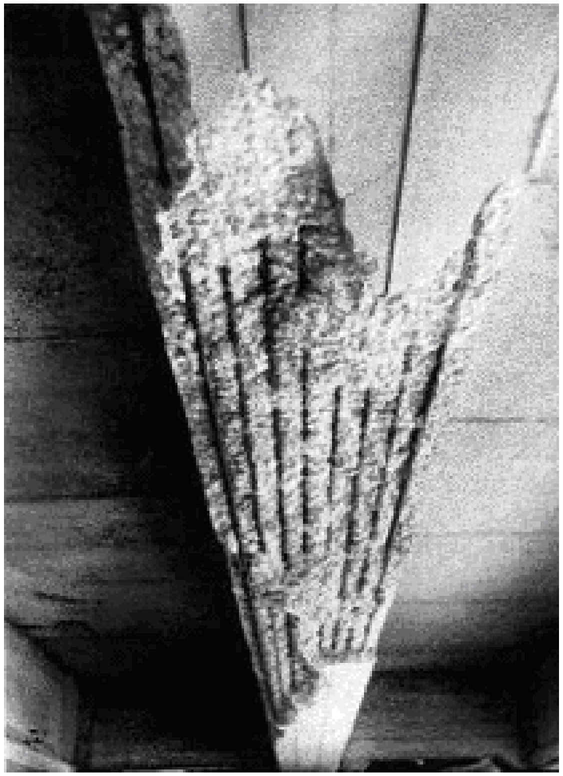
Visual Damage:
Ranking: 3.33/10

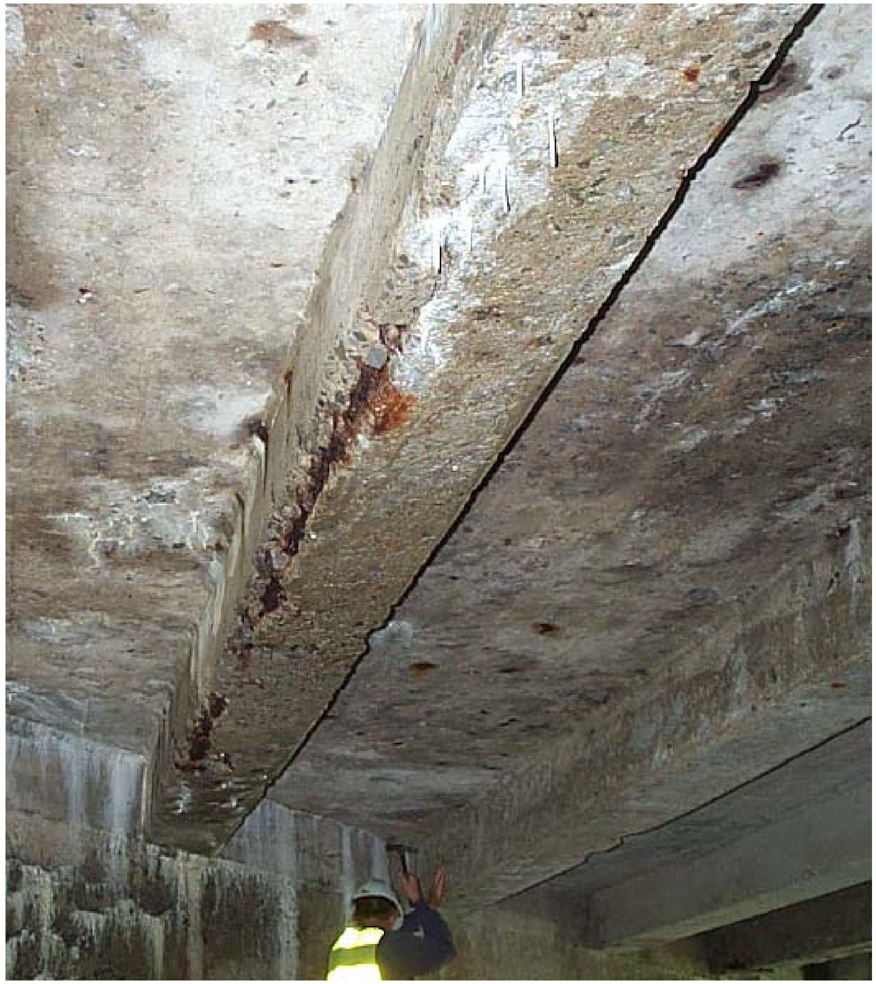
Number of Spans: 1

Status: Open

Road: SR 3008

Example 1.C.1.7 Tioga County #58-1002-0050-000
Class #3: Overall Score (30.1/100)





Details:

Span (ft): 27
Ranking: 3/10

Year Built: 1917
Ranking: 1/10

ADT: 568
Ranking: 4/10

ADTT: 57
Ranking: 3/10

Visual Damage:
Ranking: 3.33/10

Number of Spans: 1

Status: Open

Road: SR 1002

Photo on left: NCHRP Report 514, an example of a T-beam suitable for FRP repair techonology.

Photo on right: Provided by PennDOT for bridge #58-1002-0050-0000

Note: This is a very old bridge which does not carry much traffic. Also, the damage to this bridge seems to be extensive. However, this damange does not seem to closely resemble the damage described in NCHRP Report 514. Replacement of this bridge may be a better alternative to the FRP technology.

I.C.1.2.3 Visual Examples Illustrating Levels of Damage

This section further illustrates the subjective approach to classify the level of damage into: high (score = 10/10; rating = 3), moderate (score = 6.67/10; rating = 2), low (score = 3.33/10; rating = 1), and no damage (score = rating = 0). Examples I.C.1.8 through I.C.1.11, respectively, show example photographs for the four levels of damage used for the initial classification of District-3 PennDOT bridges.

Example 1.C.1.8: Visual Damage Score 10/10

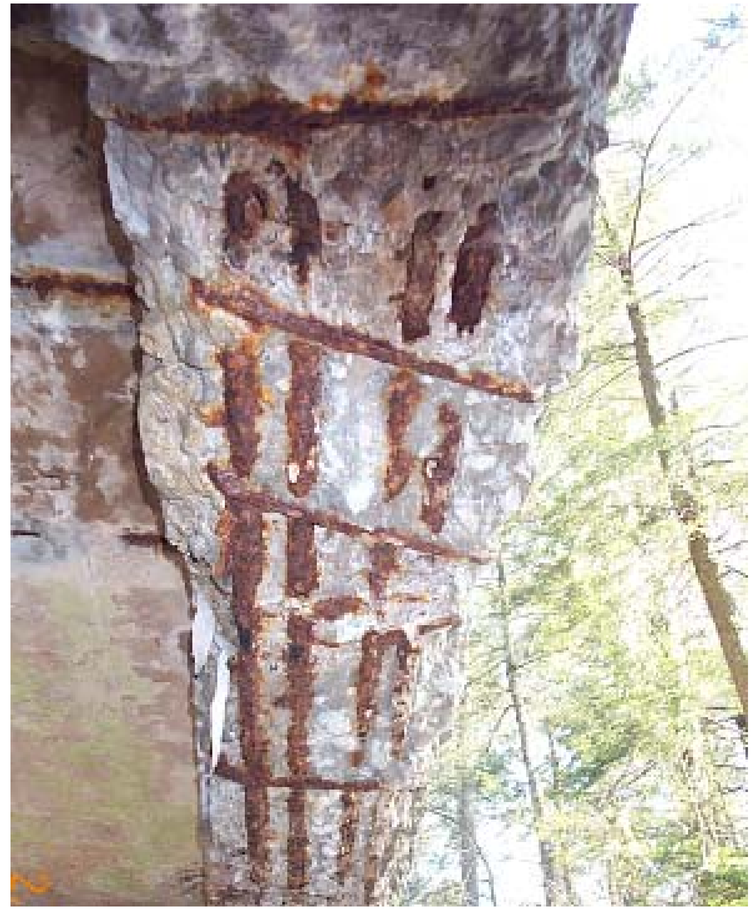


Photo on left: Provided by PennDOT for bridge #08-4013-0250-1496

Photo on right: Provided by PennDOT for bridge #59-1001-0110-2058

Note: The photos provided show extensive damage to the concrete T-beams. This type and extent of damage is suitable for FRP technology. Bridges with damage type and severity similar to this received a score of 10 out of 10 for visual damage.

Example 1.C.1.9: Visual Damage Score 6.67/10

Photo on left: Provided by PennDOT for bridge #19-0239-0060-0511

Photo on right: Provided by PennDOT for bridge #19-3008-0100-0039

Note: The damage level shown in these photos indicated that these bridges are moderate candidates for repair using the FRP technology. Bridges with damage severity similar to these examples were scored 6.67 out of 10.

Example 1.C.1.10: Visual Damage Score 3.33/10

Photo on left: Provided by PennDOT for bridge #58-1002-0050-0000

Photo on right: Provided by PennDOT for bridge #56-0220-0530-0988

Note: Some of the photographs as shown above indicated low level of damage. Damage of this type may be suited for repair by either FRP technology or conventional methods, but these bridges are not likely candidates for the FRP technology.

Example 1.C.1.11: Visual Damage Score 0/10

Photo on left: Provided by PennDOT for bridge
#59-1003-0140-0921

Photo on right: Provided by PennDOT for bridge
#59-0045-0440-1302

Note: Some of the photographs showed very little or no damage. Bridges of this type were scored 0/10 in the visual damage category, because these bridges in general probably do not require repair.

Sub-Task I-C.2: Field Visitation and Inspection

On May 2 and 3, 2005, the WVU team visited PennDOT District 3-0 to discuss the status of the project. Also, during this visit, the WVU team, guided by Jeffrey Levan of District 3-0, visited 16 concrete T-Beam Bridges. The WVU team used information gathered on this visit to re-evaluate the classification of the bridges based on visual damage, and also to define an ideal candidate bridge or bridges for repair using the FRP technology.

I-C.2.1 Bridges Visited

The WVU team visited 16 concrete T-Beam Bridges, which were chosen based on initial damage classification and their proximity to the District 3-0 office. The WVU team made general observations and took photos of each bridge in order to document the overall damage conditions and the suitability for repair using the FRP technology. Table I.C.2.1 shows the bridges which were visited by the WVU team with initial classification information.

Table I.C.2.1. Bridges Visited by the WVU Team with Initial Classification Information.

	Bridge	Span	Yr	ADT	ADTT	Visual	Rank	Class
1	41-0118-0020-1109	31	30	5600	410	3	79.8	1
2	41-0118-0030-1331	39	30	5600	410	2	67.4	2
3	41-0118-0290-0000	41	43	2600	300	2	67.7	2
4	19-0442-0050-0264	37	31	2100	100	3	70.3	1
5	41-2001-0130-0652	27	37	2600	80	2	56.4	2
6	41-2005-0052-0000	39	25	2400	210	3	70.5	1
7	59-1001-0110-2058	32	21	1400	80	3	62.8	2
8	59-0304-0060-1859	45	41	3011	90	3	77.0	1
9	59-0045-0310-2011	30	38	10300	650	3	84.8	1
10	59-2001-0020-0000	33	32	440	20	3	57.0	2
11	59-0045-0430-1068	29	37	10300	650	2	70.4	1
12	59-0015-0091-1230	50	30	9900	1300	2	73.9	1
13	54-0522-0230-1430	24	27	6300	400	2	62.9	2
14	54-0522-0210-0000	39	29	6200	400	3	79.3	1
15	54-2010-0040-1803	25	37	120	10	2	40.7	3
16	54-2013-0010-0000	23	22	200	20	3	50.3	2

Note that in this sub-task we are concerned only with the damage rating of the bridges, based on visual observations, while the other characteristics remain as defined previously (span, year, ADT/ADTT). This visual ranking is still defined as: Rating of: 3 = high or best candidate; 2 = moderate or possible candidate; 1 = low or not likely candidate for repair by the FRP technology; and 0 = not applicable for repair.

I-C.2.2: General Observations

Upon viewing the bridges, the WVU team realized that some of the damages indicated in the initial photographs provided by District 3-0 were not representative of the overall damages of the bridges. The team found that the global damages of the bridges were generally less severe than indicated by photographs of local damages. It was observed that, in many cases, the exterior beams were in a state of disrepair as indicated by the initial photos; however, the interior beams were often in good condition. With these findings, the WVU team obtained more extensive photos from the PennDOT District 3-0 library, to have a better overall representation of damages of the bridges. Using field observations and more complete photographical data of damages, the WVU team re-evaluated the classifications of the bridges in order to facilitate defining an ideal candidate bridge for repair using the FRP technology.

I-C.2.3: Re-evaluation of Criteria for Ranking Based on Visual Damages

From the observations and new data, the WVU team developed a revised ranking system for bridges in order to determine whether the FRP technology is a suitable and an economical choice for repair. This ranking is based on the extent and type of damage, both locally and globally. The original ranking was based solely on the local damages indicated by the initial photographs. The revised ranking of visual damage is defined as follows:

- Type 1: Bridges which are ranked as 1 have minimal amount of damage, both locally and globally. These bridges can be repaired using the FRP technology; however conventional repair may be more cost-effective.

- Type 2: Bridges which are ranked as 2 have moderate to severe localized damage, but overall the bridge may be in good condition. Also, bridges in this ranking may have minor to moderate damage globally. Bridges in this classification can be repair using the FRP technology; however, an assessment of overall damage and cost-effectiveness should be performed to decide whether the FRP technology is favorable over conventional methods of repair.

- Type 3: Bridges which are ranked as 3 have moderate to severe damage, both locally and globally. Due to the extent of the damage, these bridges are considered prime candidates for the FRP technology.

- Bridges which do not have any visible damage are ranked as 0 for the purpose of classification.

Using this new ranking, the bridges that were visited were re-evaluated and their new visual rankings are shown in Table I.C.2.2. As an illustration of the usefulness of the new visual damage ranking, Bridge Reference Number 9, which ranked as "3" both previously and presently, is a good candidate for repair using the FRP technology. On the other hand, Bridge Reference Number 15 was ranked "2" before as possible candidate, but it is now ranked as "1", or not likely candidate for the FRP technology. Four examples are provided (Examples I.C.2.1- I.C.2.4) to illustrate the revised visual damage ranking of bridges listed in Table I.C.2.2. Also, an example of a bridge which may be damaged beyond economical repair using the FRP technology is provided (Example I.C.2.5).

I.C.2.4: Overall Re-classification of Bridges

Based on the field visitation of 16 bridges and more extensive photographs for all of the 128 bridges included in the study, the three tentative groups defined before in Section I-C.1.1.6 are reclassified. The three categories defined before are maintained: Class 1 – Prime, Class 2 – Moderate, and Class 3 – Low; however the scores for each category have changed slightly due to subjectively observing clusters of data within distinct ranges of ranking: Group 1, 100 to 68%, Group 2, 68 to 50%, and Group 3, 50 to 0%. The revised classification of bridges is as follows:

Table I.C.2.2. New Visual Damage Rankings of Visited Bridges

Ref. No	Bridge	New Visual Ranking	Original Visual Ranking	Notes
1	41-0118-0020-1109	2	3	Exterior beam damage only
2	41-0118-0030-1331	2	2	Exterior beam damage w/min damage elsewhere
3	41-0118-0290-0000	2	2	Exterior beam damage only
4	19-0442-0050-0264	2	3	Exterior beam damage only
5	41-2001-0130-0652	3	2	All beams damaged, underside of deck damaged
6	41-2005-0052-0000	3	3	All beams damaged
7	59-1001-0110-2058	2	3	Exterior beam damage w/min damage elsewhere
8	59-0304-0060-1859	1	3	Minor damages to beams and deck
9	59-0045-0310-2011	3	3	Damage to exterior and interior beams
10	59-2001-0020-0000	2	3	Minor damages on exterior and interior beams
11	59-0045-0430-1068	3	2	Damage to exterior and interior beams
12	59-0015-0091-1230	1	2	Minor damage to deck, beams in good condition
13	54-0522-0230-1430	2	2	Exterior beam damage, interior damage minor
14	54-0522-0210-0000	2	3	Exterior beam damage, interior damage minor
15	54-2010-0040-1803	1	2	Minor exterior and minimal interior damage
16	54-2013-0010-0000	3	3	Damage to exterior and interior beams

Class 1: <u>Prime Candidate for Repair (Score of 68-100%)</u>. This set of bridges is likely a prime candidate for the FRP technology. Field investigation is suggested for the following purpose: On some of these bridges, the damage may be so severe that the most economical course of action may be to replace these structures. An ideal candidate bridge will likely be chosen from this set of bridges.

Class 2: <u>Moderate Candidate for Repair (Score of 50-68%)</u>. These bridges are likely candidates for repair. However, field investigation of these bridges is suggested to closely examine the type and extend of damage, as well as the cost-benefit of applying the FRP technology to these bridges.

Class 3: <u>Low Candidate for Repair (Score of 0-50%)</u>. Because of age, size, level of traffic, damage type or lack of damage, these bridges are not the prime focus for the FRP technology. Some of these bridges may benefit from FRP-repair technology, but they may not be the most economical choice for this type of repair.

The new classification given above has resulted in the percentages provided in Table I.C.2.3. Furthermore, the number of bridges in each damage ranking category is provided in Table I.C.2.4.

Table I.C.2.3. Number of Bridges in Each Rating Class

Rating Class	Score	No of Bridges	Percentage of Total
1	68-100	13	10.2
2	50-68	44	34.4
3	0-50	71	55.5

Table I.C.2.4. Number of Bridges in Each Damage Ranking Category

Damage Ranking	No of Bridges	Percentage of Total
3	15	11.7
2	44	34.4
1	29	22.7
0	40	31.3

Compared to the initial classification (see Fig. I.C.1.1), the number of bridges in Class 1 has decreased by 6 bridges. The number of bridges in Class 2 has decreased by 1 bridge. Consequently, the number of bridges in Class 3 has increased by 7. The details of the overall revised rating (including the weighted values as described in section I.C.1.1.5), and individual damage scores for each bridge (including general notes to describe the rationale for each rating) are provided in Tables I.C.2.5 and I.C.2.6 respectively.

Example 1.C.2.1: New Visual Damage Ranking of “1”

Bridge# 59-0304-0060-1859 - Reference No 8, Table I.C.2.1

Bridges which are ranked as 1 have minimal amount of damage, both globally and locally. These bridges can be repaired using the FRP technology; however conventional repair may be more cost-effective.

This is an example of a Type 1 ranking. Though this bridge has damage to the beams and deck, the damage is not severe. Also, several of the beams are not damaged at all.

Example 1.C.2.2: New Visual Damage Ranking of "2"

Bridge #59-2001-0020-0000 - Reference No 10 , Table I.C.2.1

Bridges which are ranked as 2 have moderate to severe localized damage. Overall the bridge is in good condition. Bridges in this classification can be repaired using the FRP technology; however, an assessment of overall damage and cost-effectiveness should be conducted to decide whether the FRP technology is favorable over conventional methods of repair.

This is an example of a Type 2 ranking. This bridge has moderately damaged exterior beams, however the rest of the beams and deck are in good condition.

Example 1.C.2.3: New Visual Damage Ranking of "3"

Bridge #41-2001-0130-0652 - Reference No 5 , Table I.C.2.1

Bridges which are ranked as 3 have moderate to severe damage, both locally and globally. Due to the extent of the damage, these bridges are prime candidates for the FRP technology.

This bridge is a good example of a Type 3 ranking. There is severe damage to both exterior and several interior beams. Also, there is severe damage to the deck. This may be a good candidate bridge for FRP repair. Due to the extent of damage, the repair cost should be considered in relation to replacement cost.

Example 1.C.2.4: New Visual Damage Ranking of "3"

Bridge #59-0045-0310-2011 - Reference No 9 , Table I.C.2.1

Bridges which are ranked as 3 have moderate to severe damage, both locally and globally. Due to the extent of the damage, these bridges are prime candidates for the FRP technology.

This bridge is a good example of a Type 3 ranking. Both exterior and interior beams have extensive damages. This bridge would be a good candidate for repair using the FRP technology.

Example 1.C.2.5: A Bridge Beyond Repair

Bridge #19-4003-0010-0082

This is an example of a bridge which has experienced extensive damage to both the concrete and the rebar. This bridge may be better suited for replacement, rather than repair using either the FRP technology or conventional methods.

Table I.C.2.5 Overall Classification of Bridges in District 3-0

Class 1 Prime Candidate for Repair: (Total = 13)

Bridge	Span	Span SC	Yr	Age SC	ADT	ADT SC	ADTT	ADTT SC	Visual	V SC	Rank	Description
59-0045-0310-2011	30	4	38	5	10300	10	650	9	3	10.0	84.8	Extensive overall damage
59-0045-0430-1068	29	3	37	5	10300	10	650	9	3	10.0	83.8	Extensive overall damage
41-0220-0131-1268	45	7	41	6	11000	10	1100	10	2	6.7	77.7	Extensive local damage
41-2014-0380-0000	23	2	36	5	6900	9	340	7	3	10.0	77.5	Extensive and moderate overall damage
49-0405-0260-0942	27	3	30	4	5700	9	110	5	3	10.0	73.5	Extensive overall damage
19-1014-0052-0442	29	3	38	5	3300	8	100	5	3	10.0	73.3	Extensive overall damage
49-4012-0250-1032	48	7	34	4	2200	6	100	5	3	10.0	72.3	Extensive to moderate overall damage.
41-0118-0230-0644	80[a]	10	43	7	2600	7	300	7	2	6.7	71.7	Extensive local damage
41-2005-0052-0000	39	5	25	3	2400	6	210	6	3	10.0	70.5	Extensive overall damage
41-2001-0130-0652	27	3	37	5	2600	7	80	4	3	10.0	69.8	Extensive overall damage
19-4003-0010-0082	174[b]	10	23	2	1200	5	80	4	3	10.0	68.8	Extensive damage to rebars, replace?
58-4024-0110-0000	46	7	34	4	650	4	110	5	3	10.0	68.8	Extensive overall damage

a. Two-span bridge, with overall span of 80 feet.
b. Three-span bridge, with overall span of 174 feet.

Table I.C.2.5 (Continued)

Class 2 Moderate Candidate for Repair: (Total = 44)

Bridge	Span	Span SC	Yr	Age SC	ADT	ADT SC	ADTT	ADTT SC	Visual	V SC	Rank	Description
41-0118-0290-0000	41	6	43	7	2600	7	300	7	2	6.7	67.7	Extensive local damage
41-0118-0030-1331	39	5	30	4	5600	9	410	8	2	6.7	67.4	Extensive local damage
41-0118-0020-1109	31	4	30	4	5600	9	410	8	2	6.7	66.4	Extensive local damage
49-0061-0016-1066	62[a]	9	34	4	5400	9	160	5	2	6.7	66.2	Extensive local damage
41-0654-0210-1810	48	7	34	4	6900	9	200	6	2	6.7	65.9	Extensive local damage
54-0522-0210-0000	39	5	29	3	6200	9	400	8	2	6.7	65.9	Extensive local damage
41-0014-0220-0000	27	3	30	4	3200	8	600	9	2	6.7	65.4	Moderate overall damage
54-0522-0090-1932	55[b]	8	29	3	4200	8	300	7	2	6.7	65.4	Moderate local damage
41-0014-0260-0368	34	4	28	3	3200	8	600	9	2	6.7	64.9	Moderate local damage
49-0225-0180-0000	66	9	27	3	3000	8	200	6	2	6.7	64.7	Moderate local damage
19-1025-0050-0000	41	6	41	6	3700	8	110	5	2	6.7	64.4	Extensive local damage
58-0414-0350-0000	31	4	41	6	2500	7	300	7	2	6.7	64.2	Minor local damage
41-0442-0160-0000	97[c]	10	27	3	3800	8	130	5	2	6.7	63.9	Extensive local damage
59-0304-0060-1859	45	7	41	6	3011	8	90	4	2	6.7	63.7	Moderate local damage
54-0522-0230-1430	24	2	27	3	6300	9	400	8	2	6.7	62.9	Extensive local damage
19-0239-0060-0511	33	4	26	3	1800	6	40	2	3	10.0	62.5	Extensive overall damage
49-1039-0020-0000	30	4	30	4	500	4	50	3	3	10.0	62.3	Extensive overall damage
56-0220-0550-0000	30	4	25	3	3400	8	340	7	2	6.7	61.4	Moderate local damage
58-4002-0020-0000	33	4	35	5	3700	8	120	5	2	6.7	60.9	Extensive local damage
59-0015-0091-1230	50	8	30	4	9900	9	1300	10	1	3.3	60.6	Minor overall damage
19-0042-0690-1821	29	3	55	10	5400	9	350	7	1	3.3	59.3	Minor local damage

a. Two-span bridge with overall span length of 62 feet.
b. Two-span bridge with overall span length of 55 feet.
c. Two-span bridge with overall span length of 97 feet.

Class 2 (continued)

Bridge	Span	Span SC	Yr	Age SC	ADT	ADT SC	ADTT	ADTT SC	Visual	V SC	Rank	Description
59-0045-0350-0000	27	3	40	6	10300	10	650	9	1	3.3	58.6	Minor overall damage
58-0249-0322-0000	51	8	36	5	1260	5	80	4	2	6.7	57.9	Moderate local damage
19-4006-0082-0000	48[a]	7	34	4	1000	5	150	5	2	6.7	57.2	Extensive local damage
58-0249-0062-0000	48	7	32	4	1500	6	90	4	2	6.7	57.2	Extensive local damage
08-4034-0140-1580	50	8	52	9	350	3	35	2	2	6.7	56.9	Extensive local damage
19-0442-0050-0264	37	5	31	4	2100	6	100	5	2	6.7	56.9	Extensive local damage
08-0414-0200-0000	30	4	26	3	1500	6	200	6	2	6.7	56.2	Extensive damage to ext beam, damage to deck
19-2009-0010-0594	41	6	30	4	2500	7	70	3	2	6.7	56.2	Extensive local damage
58-2014-0152-0000	34	4	37	5	100	1	7	1	3	10.0	55.0	Extensive overall damage
49-0225-0380-0000	29	3	34	4	1500	6	130	5	2	6.7	54.9	Moderate local damage
08-0014-0580-0000	27	3	57	10	2500	7	250	6	1	3.3	54.1	Minor damages
41-0014-0100-0000	43	6	28	3	4000	8	600	9	1	3.3	53.6	Minor damages
08-4013-0250-1496	25	3	27	3	1600	6	160	5	2	6.7	53.4	Extensive local damage
54-0522-0140-0000	67[b]	9	29	3	4200	8	300	7	1	3.3	53.1	Minor damages to deck
19-0093-0110-1085	27	3	73	10	3900	8	75	4	1	3.3	52.3	Extensive local damage
49-0054-0090-0835	25	3	59	10	3000	8	90	4	1	3.3	52.3	Minor damages
49-3010-0110-0000	48	7	31	4	636	4	60	3	2	6.7	51.9	Extensive local damage
49-0642-0130-0000	29	3	30	4	2200	6	70	3	2	6.7	51.4	Moderate local damage
49-2002-0010-1145	44	6	35	5	740	4	40	2	2	6.7	50.7	Extensive local damage
58-0015-0820-2041	37	5	42	7	12300	10	2100	10	0	0.0	50.5	No visible damage
19-4036-0012-0000	48	7	26	3	500	4	60	3	2	6.7	50.4	Extensive local damage
54-2013-0010-0000	23	2	22	2	200	2	20	1	3	10.0	50.3	Extensive overall damage
49-4018-0010-0649	27	3	22	2	3400	8	40	2	2	6.7	50.2	Extensive local damage

a. Two-span bridge with overall span length of 48 feet.
b. Two-span bridge with overall span length of 67 feet.

Table I.C.2.5 (Continued)

Class 3 Low Candidate for Repair: (Total = 71)

Bridge	Span	Span SC	Yr	Age SC	ADT	ADT SC	ADTT	ADTT SC	Visual	V SC	Rank	Description
59-1001-0110-2058	32	4	21	2	1400	5	80	4	2	6.7	49.4	Extensive local damage
54-0522-0040-0000	35	5	27	3	3000	8	360	7	1	3.3	49.1	Minor local damage
56-0220-0530-0988	33	4	25	3	4300	8	340	7	1	3.3	48.1	Minor local damage
41-1005-0130-0000	39	5	40	6	300	3	4	1	2	6.7	47.7	Extensive local damage
58-0249-0112-2535	23	2	52	9	1500	6	90	4	1	3.3	46.3	Minor local damage
41-0414-0080-0000	25	3	37	5	300	3	30	2	2	6.7	45.9	Extensive local damage
54-2004-0030-1543	33	4	41	6	160	2	13	1	2	6.7	44.9	Moderate local damage
59-0045-0440-1302	33	4	38	5	10300	10	650	9	0	0.0	44.8	No visible damage
54-2016-0050-0000	29	3	42	7	1100	5	100	5	1	3.3	44.3	Extensive local damage
19-0042-0090-0000	33	4	40	6	2200	6	75	4	1	3.3	43.8	General overall damage, should investigate
19-1025-0050-1308	33	4	41	6	2900	7	70	3	1	3.3	43.8	Minor local damage
59-2001-0020-0000	33	4	32	4	440	3	20	1	2	6.7	43.7	Moderate local damage
58-1002-0050-0000	27	3	17	1	600	4	60	3	2	6.7	43.4	Moderate overall damage
19-4045-0030-1814	35	5	34	4	220	2	20	1	2	6.7	42.9	Extensive local damage
59-3006-0050-0282	23	2	45	7	120	1	12	1	2	6.7	42.7	Moderate local damage
59-2009-0040-0000	23	2	30	4	1500	6	200	6	1	3.3	42.3	Minor local damage
08-4031-0200-0000	23	2	29	3	360	3	35	2	2	6.7	41.9	Extensive local damage
58-2005-0220-0320	33	4	42	7	950	4	80	4	1	3.3	41.8	Minor local damage
49-3018-0120-1195	35	5	38	5	1000	5	90	4	1	3.3	41.6	Minor local damage
41-2019-0080-0000	27	3	37	5	140	1	10	1	2	6.7	40.7	Moderate local damage
54-0522-0150-1440	40	6	28	3	5200	9	400	8	0	0	40.25	No visible damage
19-3008-0100-0039	23	2	33	4	270	2	20	1	2	6.7	39.9	Minor but global damage
56-0154-0050-0000	50	8	35	5	360	3	70	3	1	3.3	39.3	Minor local damage
08-0414-0230-1008	26	3	19	1	1500	6	200	6	1	3.3	38.8	Minor damages
54-0104-0120-2462	29	3	30	4	5200	9	450	8	0	0.0	38.8	No visible damage
54-0104-0230-0000	27	3	30	4	5600	9	450	8	0	0.0	38.8	No visible damage

Class 3 (continued)

Bridge	Span	Span SC	Yr	Age SC	ADT	ADT SC	ADTT	ADTT SC	Visual	V SC	Rank	Description
54-0522-0110-0469	55[a]	8	29	3	4200	8	300	7	0	0.0	38.8	Minor damages
58-0549-0020-0000	32	4	53	9	2700	7	160	5	0	0.0	38.5	Abutment failure not applicable
54-4012-0130-0000	35	5	39	5	560	4	50	3	1	3.3	38.1	Minor local damage
58-0006-0610-0000	19	1	42	7	6500	9	260	6	0	0.0	37.8	No visible damage
58-2016-0260-0543	38	5	42	7	900	4	20	1	1	3.3	37.6	Minor overall damage
58-1022-0090-0000	31	4	38	5	300	3	30	2	1	3.3	33.6	Moderate local damage
54-0235-0150-0000	94[b]	10	30	4	1100	5	100	5	0	0.0	33.5	No visible damage
49-4001-0090-0066	25	3	41	6	2700	7	160	5	0	0.0	33.0	No visible damage
54-4012-0020-0000	25	3	38	5	460	3	40	2	1	3.3	32.6	Minor local damage
54-0235-0140-0239	78[c]	9	30	4	1100	5	100	5	0	0.0	32.5	No visible damage
58-2014-0040-0946	38	5	59	10	1200	5	40	2	0	0.0	32.3	No visible damage
58-2014-0050-0815	38	5	59	10	1200	5	40	2	0	0.0	32.3	No visible damage
54-4003-0030-0254	44	6	48	8	1100	5	50	3	0	0.0	32.0	No visible damage
58-0414-0430-0190	42	6	57	10	330	3	50	3	0	0.0	31.5	No visible damage
49-3010-0160-0263	84[d]	10	40	6	750	4	70	3	0	0.0	31.3	No visible damage
58-2014-0092-0000	50	8	34	4	100	1	7	1	1	3.3	30.8	Minor overall damage
54-0235-0120-0094	43	6	30	4	1100	5	100	5	0	0.0	29.5	No visible damage
58-0006-0020-0000	67[e]	9	24	2	3500	8	32	2	0	0.0	29.5	No visible damage
58-0414-0110-0261	36	5	57	10	310	3	40	2	0	0.0	28.8	No visible damage
54-2010-0040-1803	25	3	37	5	120	1	10	1	1	3.3	27.3	Minor local damage
59-3001-0010-1018	30	4	25	3	200	2	15	1	1	3.3	27.1	Minor local damage
58-1026-0040-0304	35	5	59	10	300	3	15	1	0	0.0	27.0	No visible damage
58-3014-0120-0000	27	3	57	10	350	3	35	2	0	0	26.75	No visible damage
41-2083-0040-1285	26	3	34	4	140	1	10	1	1	3.3	25.8	Minor damages
49-4006-0160-0589	27	3	60	10	480	3	5	1	0	0.0	25.0	No visible damage

a. Two-span bridge with overall span length of 55 feet.
b. Two-span bridge with overall span length of 94 feet.
c. Two-span bridge with overall span length of 78 feet.
d. Two-span bridge with overall span length of 84 feet.
e. Two-span bridge with overall span length of 67 feet.

Class 3 (continued)

Bridge	Span	Span SC	Yr	Age SC	ADT	ADT SC	ADTT	ADTT SC	Visual	V SC	Rank	Description
58-0414-0540-1036	19	1	54	9	330	3	50	3	0	0.0	25.0	No visible damage
54-3012-0080-0000	27	3	37	5	520	4	80	4	0	0.0	24.5	No visible damage
49-4022-0010-1591	23	2	22	2	230	2	10	1	1	3.3	23.6	Minor local damage
49-2001-0020-0000	23	2	28	3	120	1	10	1	1	3.3	23.3	Minor damages
58-3006-0060-3546	33	4	50	9	150	2	15	1	0	0.0	22.8	No visible damage
54-2012-0090-0091	29	3	51	9	210	2	2	1	0	0.0	21.8	No visible damage
49-3011-0010-0238	27	3	57	10	70	1	6	1	0	0.0	21.5	Minor damages
56-0154-0150-1564	26	3	35	5	360	3	70	3	0	0.0	21.0	No visible damage
56-4008-0080-0000	30	4	50	9	120	1	10	1	0	0.0	21.0	No visible damage
49-3003-0020-0645	24	2	22	2	1000	5	90	4	0	0.0	20.8	No visible damage
59-1003-0140-0921	35	5	36	5	385	3	10	1	0	0.0	19.5	No visible damage
54-2003-0020-0000	23	2	49	8	150	2	6	1	0	0.0	19.3	Minor local damage
56-4020-0010-0000	26	3	49	8	140	1	10	1	0	0.0	18.5	No visible damage
56-1010-0030-0980	19	1	51	9	110	1	10	1	0	0.0	18.0	No visible damage
56-1002-0110-0000	22	2	41	6	100	1	10	1	0	0.0	14.5	No visible damage
56-1002-0140-0000	15	1	40	6	100	1	10	1	0	0.0	13.5	No visible damage
49-4020-0010-1014	31	4	21	2	230	2	20	1	0	0.0	12.3	No visible damage
49-2005-0040-1415	24	2	30	4	130	1	10	1	0	0.0	11.5	No visible damage
58-1004-0120-1913	27	3	19	1	300	3	9	1	0	0	11.5	No visible damage
54-2004-0020-0000	23	2	22	2	160	2	13	1	0	0	10.25	No visible damage

Table I.C.2.6 Ranking of Bridges Based on Damage Only

Bridge Damage Ranking of 3: Moderate to Severe Overall Damage (Total = 15)

Bridge	Visual	V SC	Description
41-2014-0380-0000	3	10.0	Extensive and moderate overall damage
59-0045-0310-2011	3	10.0	Extensive overall damage
59-0045-0430-1068	3	10.0	Extensive overall damage
49-0405-0260-0942	3	10.0	Extensive overall damage
19-1014-0052-0442	3	10.0	Extensive overall damage
49-4012-0250-1032	3	10.0	Extensive to moderate overall damage.
41-2005-0052-0000	3	10.0	Extensive overall damage
41-2001-0130-0652	3	10.0	Extensive overall damage
19-4003-0010-0082	3	10.0	Extensive damage to rebars, replace?
58-4024-0110-0000	3	10.0	Extensive overall damage
19-0239-0060-0511	3	10.0	Extensive overall damage
49-1039-0020-0000	3	10.0	Extensive overall damage
58-2014-0152-0000	3	10.0	Extensive overall damage
54-2013-0010-0000	3	10.0	Extensive overall damage
41-0042-0050-1036	3	10.0	Extensive overall damage

Bridge Damage Ranking of 2: Moderate to Severe Local Damage (Total = 44)

Bridge	Visual	V SC	Description
41-0118-0230-0644	2	6.7	Severe overall damage, minor local damage, replace?
58-0414-0350-0000	2	6.7	Minor local damage
49-3010-0110-0000	2	6.7	Extensive local damage
41-0220-0131-1268	2	6.7	Extensive local damage
41-0118-0290-0000	2	6.7	Extensive local damage
41-0118-0030-1331	2	6.7	Extensive local damage
41-0118-0020-1109	2	6.7	Extensive local damage
49-0061-0016-1066	2	6.7	Extensive local damage
41-0654-0210-1810	2	6.7	Extensive local damage
54-0522-0210-0000	2	6.7	Extensive local damage
41-0014-0220-0000	2	6.7	Moderate overall damage
54-0522-0090-1932	2	6.7	Moderate local damage
41-0014-0260-0368	2	6.7	Moderate local damage
49-0225-0180-0000	2	6.7	Moderate local damage
19-1025-0050-0000	2	6.7	Extensive local damage
41-0442-0160-0000	2	6.7	Extensive local damage
59-0304-0060-1859	2	6.7	Moderate local damage
54-0522-0230-1430	2	6.7	Extensive local damage
56-0220-0550-0000	2	6.7	Moderate local damage
58-4002-0020-0000	2	6.7	Extensive local damage
58-0249-0322-0000	2	6.7	Moderate local damage
19-4006-0082-0000	2	6.7	Extensive local damage

Table I.C.2.6 (continued)

Bridge Damage Ranking of 2 (continued)

Bridge	Visual	V SC	Description
58-0249-0062-0000	2	6.7	Extensive local damage
08-4034-0140-1580	2	6.7	Extensive local damage
19-0442-0050-0264	2	6.7	Extensive local damage
08-0414-0200-0000	2	6.7	Extensive damage to ext beam, damage to deck
19-2009-0010-0594	2	6.7	Extensive local damage
49-0225-0380-0000	2	6.7	Moderate local damage
08-4013-0250-1496	2	6.7	Extensive local damage
49-0642-0130-0000	2	6.7	Moderate local damage
49-2002-0010-1145	2	6.7	Extensive local damage
19-4036-0012-0000	2	6.7	Extensive local damage
49-4018-0010-0649	2	6.7	Extensive local damage
59-1001-0110-2058	2	6.7	Extensive local damage
41-1005-0130-0000	2	6.7	Extensive local damage
41-0414-0080-0000	2	6.7	Extensive local damage
54-2004-0030-1543	2	6.7	Moderate local damage
59-2001-0020-0000	2	6.7	Moderate local damage
58-1002-0050-0000	2	6.7	Moderate overall damage
19-4045-0030-1814	2	6.7	Extensive local damage
59-3006-0050-0282	2	6.7	Moderate local damage
08-4031-0200-0000	2	6.7	Extensive local damage
41-2019-0080-0000	2	6.7	Moderate local damage
19-3008-0100-0039	2	6.7	Minor but global damage

Table I.C.2.6 (continued)

Bridge Damage Ranking of 1: Minimal Damage (Total = 29)

Bridge	Visual	V SC	Description
59-0015-0091-1230	1	3.3	Minor overall damage
19-0042-0690-1821	1	3.3	Minor local damage
59-0045-0350-0000	1	3.3	Minor overall damage
08-0014-0580-0000	1	3.3	Minor damages
41-0014-0100-0000	1	3.3	Minor damages
54-0522-0140-0000	1	3.3	Minor damages to deck
19-0093-0110-1085	1	3.3	Extensive local damage
49-0054-0090-0835	1	3.3	Minor damages
54-0522-0040-0000	1	3.3	Minor local damage
56-0220-0530-0988	1	3.3	Minor local damage
58-0249-0112-2535	1	3.3	Minor local damage
19-0042-0090-0000	1	3.3	General overall damage, should investigate
19-1025-0050-1308	1	3.3	Minor local damage
59-2009-0040-0000	1	3.3	Minor local damage
58-2005-0220-0320	1	3.3	Minor local damage
49-3018-0120-1195	1	3.3	Minor local damage
56-0154-0050-0000	1	3.3	Minor local damage
08-0414-0230-1008	1	3.3	Minor damages
54-4012-0130-0000	1	3.3	Minor local damage
58-2016-0260-0543	1	3.3	Minor overall damage
58-1022-0090-0000	1	3.3	Moderate local damage
54-4012-0020-0000	1	3.3	Minor local damage
58-2014-0092-0000	1	3.3	Minor overall damage
54-2010-0040-1803	1	3.3	Minor local damage
59-3001-0010-1018	1	3.3	Minor local damage
41-2083-0040-1285	1	3.3	Minor damages
49-4022-0010-1591	1	3.3	Minor local damage
49-2001-0020-0000	1	3.3	Minor damages
54-2016-0050-0000	1	3.3	Extensive local damage

Table I.C.2.6 (continued)

Bridges with No Visual Damage (Total = 40)

Bridge	Visual	V SC	Description
58-0015-0820-2041	0	0.0	No visible damage
59-0045-0440-1302	0	0.0	No visible damage
54-0522-0150-1440	0	0.0	No visible damage
54-0104-0120-2462	0	0.0	No visible damage
54-0104-0230-0000	0	0.0	No visible damage
54-0522-0110-0469	0	0.0	Minor damages
58-0549-0020-0000	0	0.0	Abutment failure not applicable
58-0006-0610-0000	0	0.0	No visible damage
54-0235-0150-0000	0	0.0	No visible damage
49-4001-0090-0066	0	0.0	No visible damage
54-0235-0140-0239	0	0.0	No visible damage
58-2014-0040-0946	0	0.0	No visible damage
58-2014-0050-0815	0	0.0	No visible damage
54-4003-0030-0254	0	0.0	No visible damage
58-0414-0430-0190	0	0.0	No visible damage
49-3010-0160-0263	0	0.0	No visible damage
54-0235-0120-0094	0	0.0	No visible damage
58-0006-0020-0000	0	0.0	No visible damage
58-0414-0110-0261	0	0.0	No visible damage
58-1026-0040-0304	0	0.0	No visible damage
58-3014-0120-0000	0	0.0	No visible damage
49-4006-0160-0589	0	0.0	No visible damage
58-0414-0540-1036	0	0.0	No visible damage

Bridges with No Visual Damage (continued)

Bridge	Visual	V SC	Description
54-3012-0080-0000	0	0.0	No visible damage
58-3006-0060-3546	0	0.0	No visible damage
54-2012-0090-0091	0	0.0	No visible damage
49-3011-0010-0238	0	0.0	Minor damages
56-0154-0150-1564	0	0.0	No visible damage
56-4008-0080-0000	0	0.0	No visible damage
49-3003-0020-0645	0	0.0	No visible damage
59-1003-0140-0921	0	0.0	No visible damage
54-2003-0020-0000	0	0.0	Minor local damage
56-4020-0010-0000	0	0.0	No visible damage
56-1010-0030-0980	0	0.0	No visible damage
56-1002-0110-0000	0	0.0	No visible damage
56-1002-0140-0000	0	0.0	No visible damage
49-4020-0010-1014	0	0.0	No visible damage
49-2005-0040-1415	0	0.0	No visible damage
58-1004-0120-1913	0	0.0	No visible damage
54-2004-0020-0000	0	0.0	No visible damage

Sub-Task I-C.3: Recommendations for Selecting Candidate Bridges

This section presents recommendations for selecting candidate bridges that have the greatest potential for demonstrating the benefits of surface applied FRP for repairing damaged concrete bridge components. Considering the literature review (Task I.A) and survey results (Task I.B), the recommendations in this section are based on the classification approach described in Sub-task I-C.1, leading to the subsequent ratings of all bridges in Sub-task I-C.2, with results corroborated by field observations of a selected group of 16 bridges.

The selection of Candidate Bridges for FRP-repair can be based on the following two aspects: (1) considerations for selection, and (2) selection for different levels of repair efforts.

I-C.3.1 Considerations for Selection of Candidate Bridges

The considerations for selecting candidate bridges should include: (1) a rational procedure for identifying suitable candidate bridges, (2) a preliminary analysis method for performance and cost effectiveness, and (3) a detailed structural analysis and implementation plan.

I-C.3.1.1 Suitable Candidates

The rating methodology described in Subtask I-C.2, and summarized in Section I-C.2.4, identifies the list of potential candidate bridges from the general District-3 inventory. In general, structures that meet most of the following criteria are considered candidates for further investigation:

1. Age: Built between the 1920's and 1940's
2. Span: Single span structures between 20 and 50 feet
3. ADT: Any level of car traffic
4. ADTT: Any level of truck traffic, but usually about 1000 trucks per day
5. Visual Rating: Bridges that ranked either as Type 2 or Type 3

Based on the classification shown in Table I.C.2.3, approximately half of the District-3 T-Beam concrete bridge inventory scored 50% or greater, indicating that they can be considered suitable candidates for FRP-repair technology. In the classification, 13 Prime Candidate Class-1 bridges had a rating of 68% to 100%, and 44 Moderate Candidate Class-2 bridges had a score ranging between 50% and 68%. The remaining 71 Low Candidate Class-3 bridges scored between 0% and 50%. This indicates that about half of the bridges are classified as being prime or likely candidates for FRP repair, and they can be further inspected for visual condition assessments to select appropriate structures for implementation of FRP repair.

Out of a total of 128, initially 61 bridges were identified as suitable candidates for FRP repair, and out of these 16 were further selected for visual inspection. The results and reclassification of these structures are explained in detail in Sub-task I-C.2, resulting in a final selection of 57 bridges as suitable candidates for FRP repair. During the visual inspection, a few of the structures in District-3 inventory were immediately considered to be better suited for replacement than for repair. In general, however, the visual inspection either confirmed the initial condition assessment of the structure or provided additional information as to the scope of possible repair. The anticipated scope of repair for all 128 bridges ranges from simple maintenance to possible replacement, and includes the following scenarios:

1. General cleaning and maintenance
2. Simple patching
3. Patching and use of surface bonded FRP
4. Moderate rehabilitation including concrete chipping, forming and casting, plus the use of surface bonded FRP
5. Extensive rehabilitation including chipping, forming and casting, with use of surface bonded FRP
6. Complete replacement

In summary, the candidate bridges for FRP repair should be selected from those listed in Table I.C.2.5 either as Class 1 (Prime) or Class 2 (Moderate).

I-C.3.1.2 Preliminary Analysis

After a review of the bridges that were ranked either as Prime or Moderate candidates for FRP-repair, a preliminary analysis of selected candidate bridges should be conducted, using design guidelines published by ACI - Committee 440, and other available sources, such as Missouri DOT's design guide developed by the University of Missouri-Rolla.

The first and most fundamental consideration is the design loads. Current and future anticipated loads need to be considered to verify design assumptions. The two primary assumptions are:

1. Loads are within the elastic range of the structure
2. Existing section properties are known (area of steel, yield point of steel, concrete strength and dimensions of the beam)

For a likely candidate bridge, a preliminary design can be performed to estimate the quantities of required FRP. The preliminary design will also yield sufficient knowledge of the scope of repair using conventional materials.

As-built drawings, original design sheets or further field surveys can be used to ascertain the initial data for analysis. Ultimately, field surveys to determine the actual area of steel, in-situ concrete strength, and knowledge of the damaged condition of the structure will be required for a detailed analysis.

The overall cost of repair can be determined and a cost comparison with conventional materials can be made. It is recommended that all factors be included in the cost comparison. Factors including labor, equipment, cost of lane closures, economic impact caused by detours, and future cost of replacement should be considered in addition to the acquisition cost of FRP materials.

In summary, for a given bridge selected from the recommended list of Prime or Moderate candidate bridges (Table I.C.2.5), a preliminary analysis should be performed following ACI 440. Also, this preliminary assessment should include a cost-benefit analysis as illustrated in Task-D.

I-C.3.1.3 Detailed Analysis

After a candidate structure is identified for repair using surface bonded FRP, and following a preliminary analysis, a detailed analysis of the structure should be conducted. Currently there is no analysis or design standard in any AASHTO documents to determine the type and amount of FRP required. In lieu of an AASHTO standard or guide, it is recommended that the methodology presented by the American Concrete Institute in 440.2R - Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures - be used as a basis for design.

This ACI document can guide the designer through the process of analyzing and detailing surface bonded FRP. Some of the highlights include analysis of existing structural conditions, analysis of section properties, computation and checks for applied loads, computation and checks for allowable strain, and consideration of deflection for serviceability.

One of the limitations of this approach is the lack of certainty of the actual existing conditions of the structure. It is important to determine the failure mode that will control the behavior of the repaired structural component. The possible failure modes are discussed in I-A.1.3.2 – Structural Response, with possible three primary flexural failure modes being:

1. Crushing of concrete prior to yielding of the reinforcing steel
2. Yielding of tension reinforcing steel followed by rupture of FRP
3. Yielding of steel prior to crushing of concrete

The knowledge of the failure mode is important to ensure that the structure is within the elastic range, that the behavior is ductile, and that full structural benefit of the FRP is realized (see Fig. I.A.1.2).

In summary, a detailed analysis is required for the effective implementation of FRP-repair in a selected bridge project.

I-C.3.2 Selection of Candidate Bridges for Different Levels of Repair

Analysis, design and specification of the repair can be performed either by an outside consultant/contractor, or in-house by District personnel. Depending on the overall scope of work, the FRP-repair for candidate bridges may be defined at three levels: (1) Major, with all work contracted out; (2) Moderate, with combined outside consultant and in-house personnel; and (3) Minor, with all work accomplished in-house.

The selection of candidate bridges should conform to this descending scale of FRP-repair work complexity, from Major to Moderate to Minor. This approach offers great potential for providing an efficient learning and training environment for District personnel, so that eventually, a Minor FRP-repair work of a candidate bridge can be accomplished entirely by District forces. Moreover, this approach provides an opportunity to develop more specific design and construction guidelines, leading to training seminars for PennDOT personnel.

This section describes the three proposed levels of FRP-repair, and identifies correspondingly suitable bridge examples from the recommended lists of Class 1 and Class 2 bridges, given in Table I.C.2.5.

I-C.3.2.1 Major Candidate Bridges (Level 1)

The first level should address a candidate bridge requiring extensive repair. At this level the engineering is performed by a consultant and the actual repair is contracted out through competitive bid. An example of a bridge that can be addressed by this level of repair is Bridge #41-0118-0230-0644, as presented in Subtask I-C.2.4.

At this level all of the preparatory concrete work and application of the surface bonded FRP would be performed per project specifications and construction drawings. It is expected that the overall scope of work may include other pay items such as abutment repair, bearing repair, and possibly expansion joint repair. While the application of the FRP may represent a smaller portion of the overall project cost, the scale of the project can offer sufficient opportunities for developing expertise with FRP technology within the District and PennDOT.

Because of the extensive damage in such structures, as shown in this bridge, the use of FRP for additional strengthening is possible, such as in areas deficient in shear or areas that require confinement of reinforcing steel due to low cover or inadequate splice length.

A typical project at this scale, used as a prototype demonstration structure for gaining experience with FRP technology, can include the following primary tasks: (1) field assessment of the structure, (2) structural analysis and load testing, (3) evaluation of in-situ material properties from field samples, (4) design of FRP repair and other work, (5) repair implementation and evaluations, (6) testing of repaired structure, and (7) supporting lab-scale studies, and (8) development of draft specifications and standard drawings.

I-C.3.2.2 Moderate Candidate Bridges (Level 2)

The next level of repair should address a candidate bridge showing moderate level of damage. A combination of work performed by District forces and some contracting through specialty trades or engineering by a consultant is suggested. An example of a structure that could be addressed by this level of repair is Bridge #59-0045-0310-2011, as presented in Table I.C.2.5.

At this level most field activities can be accomplished by District forces. However, because there may be a need for injecting cracks with either epoxy or urethane, or the overall cost may exceed allocated District limits, it may be necessary to advertise portions of the project for competitive bids. Several possibilities could be explored, such as: (1) retaining a consultant for the engineering portion and using District forces for labor (possibly in phases); (2) performing the engineering in-house and contracting out the specialty items (crack injection or application of the FRP), with District forces acting as a general contractor; finally (3), performing the engineering in-house and contracting out all of the field work.

At this level of repair, the District would realize the benefits of competitive bidding while retaining overall design control and providing training opportunities.

I-C.3.2.3 Minor Candidate Bridges (Level 3)

The third level should be addressed for a candidate bridge with minor to moderate level of damage. This level can be described as a trial or test program to evaluate the implementation of repair techniques by District forces on a relatively small scale. A structure that could be addressed by this level of repair is Bridge #41-0118-0020-1109, as presented in Table I.C.2.5. In general a structure with moderate and localized damage is a likely candidate for a trial program.

From a field assessment, the expected scope of work could include removing loose concrete, cleaning existing reinforcing steel, measuring the current area of steel, estimating the compressive strength of the concrete, forming and casting new concrete or mortar to re-establish the beam section, then application of surface bonded FRP after appropriate surface preparation. To expedite the project, final design to determine the quantities of the FRP can be performed while preparatory work is being completed.

At this level the anticipated scope of work, although detailed, is small and it is realistic to assume that the District could accomplish this using in-house engineering and District forces. Funding for this type of maintenance construction is realistically within the limits of a District force account.

The apparent benefits of executing work at this trial level are hands-on opportunities for training personnel and evaluating FRP-repair technology. The experiences gained at this level will naturally lead to further in-house bridge repair projects with FRP, including eventually Moderate and Major candidate bridges. Also, District personnel will develop better understanding of relevant issues that should be addressed in the development of specifications for PennDOT.

Section 4 – Task I-D: Outcome Measures

Concrete structures and particularly concrete highway bridges are exposed over time to deleterious effects of environmental attacks, leading to degradation of the material. As a consequence, concrete may delaminate at the reinforcement level, leading to cracking and spalling. Since the replacement of structures is very costly, rehabilitation techniques have been used to extend the life of deteriorating structures. Conventional techniques, such as concrete patching, have been used by state departments of transportations (DOTs) to prolong the life of structures. Worldwide deterioration of structures has motivated the development of new and more cost-effective solutions. One of these solutions is the use of externally bonded Fiber-Reinforced Polymer (FRP) composites. FRP composites in the form of fabrics and laminates have been externally bonded to concrete structures to increase structural capacity and provide longer service-life. The application of this technology in practice has been highly successful [1]. This section provides an overview of both conventional rehabilitation techniques and repair using FRP technology. Also, a comparison of the process and cost-effectiveness of the two methods is presented. Furthermore, this section provides a comparison between replacement and complete rehabilitation using the FRP technology. Example repairs, including Gantt charts and costs are provided for three examples bridges in PennDOT's District-3 inventory. Similar repairs have been conducted elsewhere, and particularly in Missouri. Examples of their bridge repairs are provided to further illustrate the applications of FRP technology.

Sub-Task I-D.1: Overview of Conventional Techniques

In an effort to prevent further corrosion of steel rebar and deterioration of reinforced concrete T-Beam bridges, a conventional repair technique has been developed and employed. This method involves: removal of all loose and unsound concrete, replacement and/or additional placement of rebar, and placement of formwork to cast a concrete cover or "jacket". Though this process seems quite simple, formwork and repair of rebar requires significant effort. Figure I.D.1.1 shows an example of a suggested formwork provided by PennDOT District-3 for a conventional T-Beam repair.

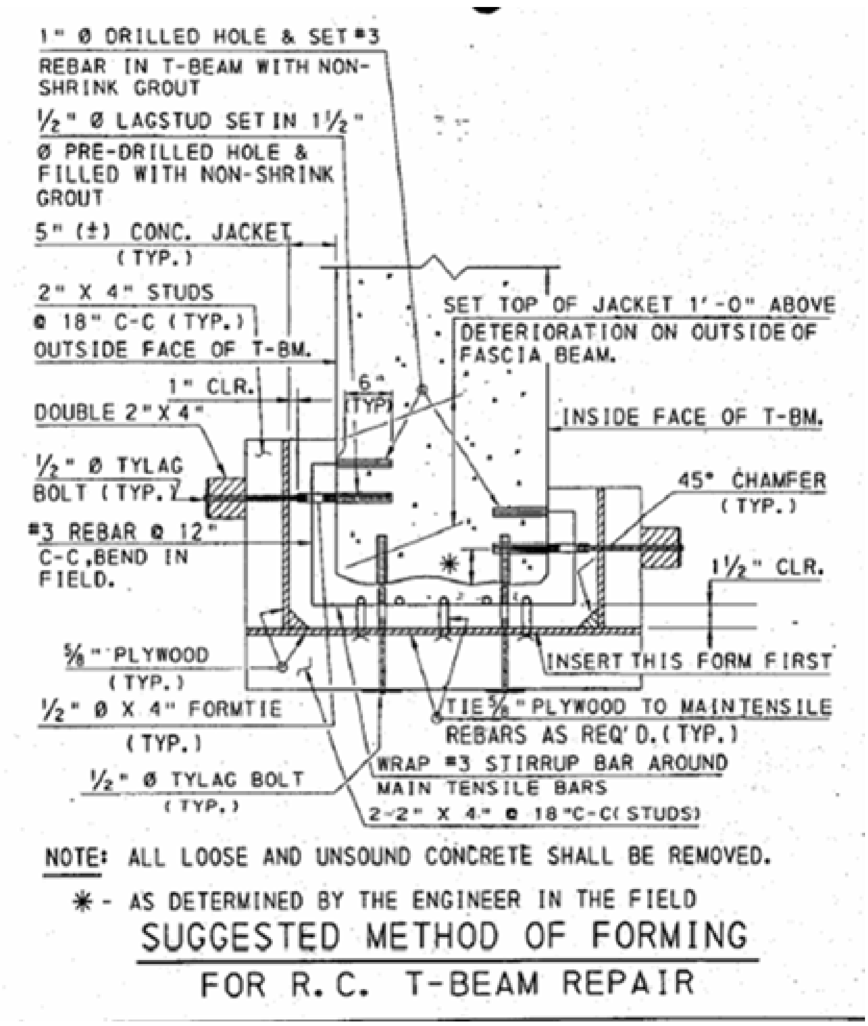


Fig. I.D.1.1. Formwork Diagram for Conventional Repair by PennDOT District-3.

Sub-Task I-D.2: Overview of Surface-Bonded FRP Technology

The behavior of concrete members retrofitted with FRP systems is highly dependent on a sound concrete substrate and proper preparation and profiling of the concrete surface. Surface preparation of the substrate is essential to achieve a good bond. The surface must be clean of all laitance, dust, dirt, oil, curing compound and existing coatings by appropriate means [1]. Other irregularities should be filled or ground smooth accordingly in order to provide an acceptable surface. Once the surface is prepared, the FRP is applied using the manufacturer's specifications. To provide an overview of the FRP surface-bonded repair process, a typical installation procedure prescribed by Fyfe Co. [2], a supplier of this technology, is as follows:

1. Prepare surface to receive the composite laminate strips by abrasive blasting or grinding to achieve 1/16th-inch minimum aggregate amplitude (International Concrete Repair Institute CSP 5 – Guideline No.03732).
2. Broom or air-clean surface per specification.
3. Apply primer coat of TYFO® S or TYFO® WS (carbon composite laminate strips) to the prepared substrate.
4. Apply a thick coat (approximately 1/16”) of TYFO® TC to the primed substrate for bond.
5. Clean the abraded side of the laminate (side without lettering) with acetone or other similar solvent to remove all foreign materials – allow complete evaporation of solvents prior to proceeding with the installation.
6. Apply TYFO® TC epoxy adhesive (approximately 1/16”) to the cleaned laminate surface using a trowel or by similar method. Note – The TYFO® TC is not to be applied to the cleaned strip for a minimum of 30 minutes to allow complete evaporation of the Acetone or other solvent.
7. Allow sufficient time for the epoxy adhesive to become tacky as required.
8. Apply strips to surface by hand placement. The laminate shall then be pressed to the substrate using a hard roller (rubber or steel) to assure both proper bond and uniform elevation. Excess extruded epoxy may be removed by trowel or similar method.
9. Clean excess epoxy from installed laminate and adjacent areas.
10. Finish: Refer to architectural specification for final finish. Use system as directed by manufacturer.
11. Cure: The composite shall be protected from contact by moisture for a period of a minimum of three days. Curing is complete after seven days.

Note: TYFO® is the carbon composite laminate strips.

I-D.2.1 Example Repair from Missouri Department of Transportation

To illustrate the application and process of the surface-bonded FRP technology, an example repair implemented in two similar bridges is provided. The repairs were performed by the Missouri Department of Transportation (MODOT) in conjunction with the University of Missouri-Rolla (UMR) under the direction of Dr. Nestore Galati. These bridges (Bridge X-0596 and Bridge T-0530) are similar in design and have experienced a similar level and type of damage as the bridges in the PennDOT District-3 inventory. Figures I.D.2.1 and I.D.2.2 show photos of Bridge X-0596, for overall view and condition of the exterior girder prior to repair, respectively. Table I-D.2.1 shows a summary of information of Bridge X-0596 which was repaired. The MODOT repairs were targeted for both rehabilitation and strengthening. Although strength gain may not be a primary objective in some projects, it is an added benefit of repair using FRP technology.

Fig. I.D.2.1 Bridge X-0596 of MODOT [3].

Fig. I.D.2.2. Condition of Exterior Girder of Bridge X-0596-MDOT [3].

Table I-D.2.1. Summary of Information of Repair of Bridge X-0596-MDOT [3].

Bridge X-0596	
	
District: 5, Missouri	**County**: Morgan
Year Built: 1946	**Featured Intersection**: Lander Fork Creek
Main Spans Construction: RC T-Beams	**Number of Main Spans**: 3
Total Length: 137.5 feet	
Bridge Features	
Geometry • Roadway width is 20 ft • Alignment is tangential • Three-span deck • Three RC T-Beams spaced 9 ft on centers • Slab thickness is 6 in.	
Concrete Conditions Prior to Application • Cracks in exterior girders, some reinforcement exposed and some concrete deteriorated • Cracks and deterioration of beams in intermediate bents • Cracks and deterioration of the ends of the girders at intermediate bents • End Bents in better condition (but with rusty steel bearing plates)	

I-D.2.1.1 Substrate Repair

The performance of the composite system not only depends on the strength and quality of the concrete substrate, but also on the bond between the composite and substrate. Unsound concrete was removed and patched. Also, holes were filled. All concrete surfaces were prepared in accordance with minimum requirement defined in the Masters Materials and Construction Specification [3].

I-D.2.1.2: Surface Preparation

To promote continuous and intimate contact between the substrate and FRP, concrete irregularities were removed and smoothed to less than 1 mm. Also, all corners were rounded to reduce stress concentrations. Finally, abrasive sandblasting was used to clean the concrete surfaces of dust, dirt, laitance, oil and any curing substances. Concrete surface roughness was equivalent to CSP 3 (Concrete Surface Profile number 3) as defined by the International Concrete Repair Institute [3].

I-D.2.1.3: Externally Bonded Composite Reinforcement

Three spans of Bridge X-0596 were bonded with manual lay-up laminates. The installation process is described next.

Epoxy primer was used to fill voids in the concrete surface. All surfaces which were to be bonded with FRP were primed with a penetrating primer. Primer was mixed in accordance with the manufacturer’s specified ratio and applied using brushes and rollers.

The carbon fiber sheets were cut (see Figure I.D.2.3, photos from Bridge T-0530) before placement into prescribed sizes, and were installed by manual lay-up methods. The sheets were saturated by rolling out the external surface. After appropriate time (10 minutes), a second saturant application was applied to complete impregnation. The saturant was applied in strict accordance with the manufacturer’s recommendations. Figure I.D.2.4 (photos from Bridge T-0530) shows manual lay-up FRP sheet installation [4].

I-D.2.1.4: Traffic Control

One of the benefits of the application of this technology is that the bridge does not need to be closed during installation. However, in order to avoid vibrations on Bridge T-0530 during the installation, traffic control was used. Speed of vehicles was limited to 15 mph.

Fig. I.D.2.3. Cutting of FRP sheets [4].

Fig. I.D.2.4. Manual Lay-up FRP Sheet Installation [4].

Sub-Task I-D.3: Comparison Between Methods

In order to compare the conventional and FRP repair methods, a sample repair scheme using both of these methods is provided for the three levels of repair suggested in Subtask 1.C.3. This repair scheme includes comparisons for both the construction process and a cost analysis. PennDOT Bridge #41-0118-0230-0644 was selected for a Level-1 repair (Example 1), Bridge #59-0045-0310-2011 was selected for a Level-2 repair (Example 2), and Bridge #41-0118-0020-1109 was selected for a Level-3 repair (Example 3). These examples are presented in Sections I-D.3.3, I-D.3.4 and I-D.3.5 respectively.

The next section (I-D.3.1) provides a comparison between construction procedures for a generalized 3-beam repair using both the FRP technology and conventional methods. Generalized design assumptions for repair using both methods are given in Section I-D.3.2.

I-D.3.1: Comparison of Construction Procedures

For both repair methods, the initial surface preparation is similar. However, due to the importance of bonding with FRP, surface preparation quality control for FRP application is stricter. The two repair methods are quite different after the initial surface preparation. Figures I.D.3.1 and I.D.3.2 show sample Gantt charts for repair of a typical 3-beam system (Example 2, Section I-D.3.4) using both conventional and the FRP technologies respectively.

Microsoft Project [5], which is project-management software, allows the user to input tasks, task durations, and task dependencies. This software was used to produce general Gantt charts, comparing construction processes for both the conventional method and the FRP technology.

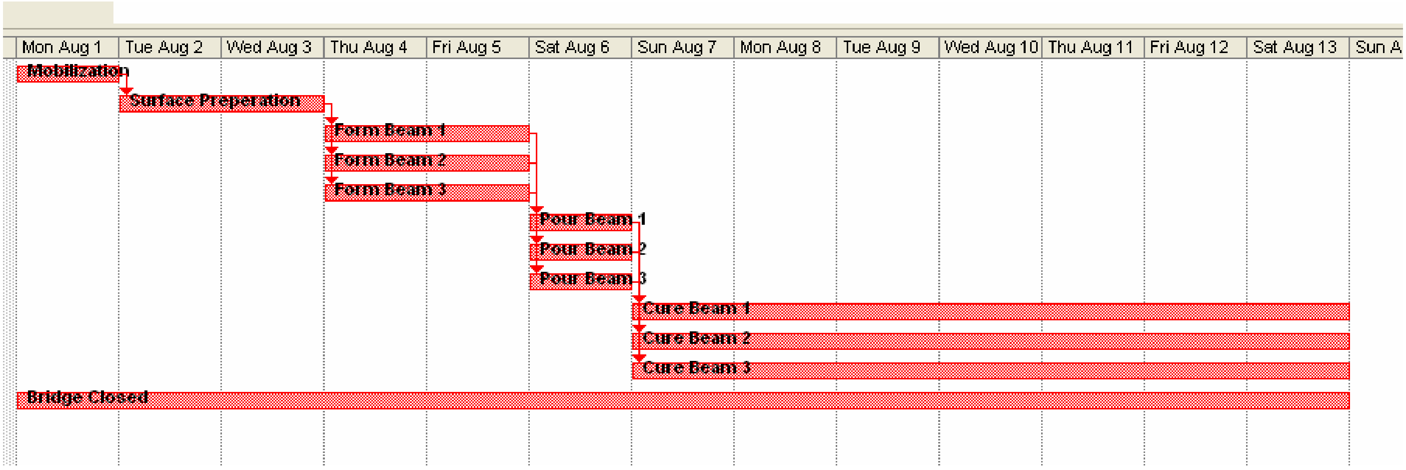


Fig. I.D.3.1. Gantt Chart for Conventional Repair (obtained with [5]).

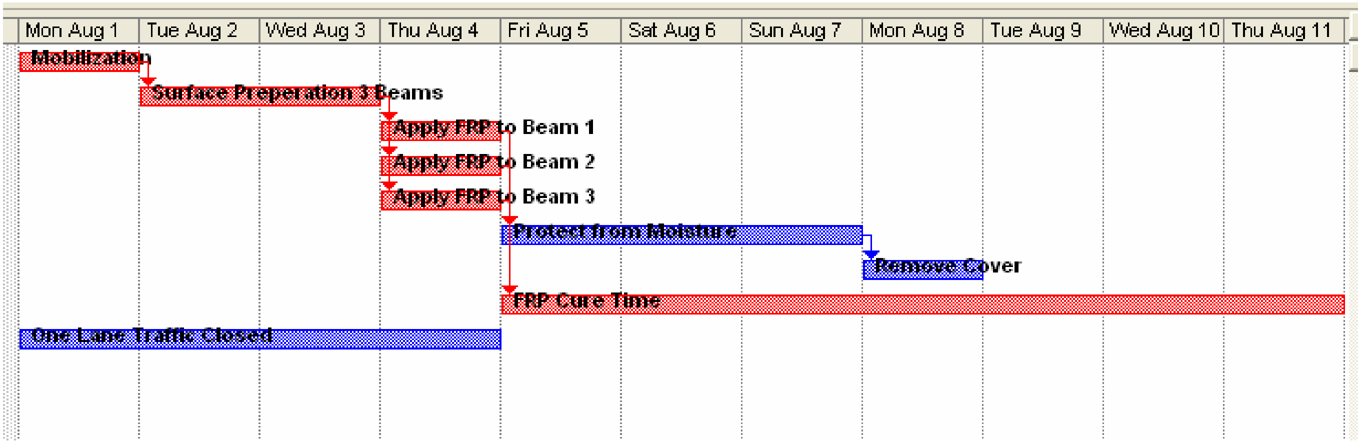


Fig. I.D.3.2. Gantt Chart for Surface-Bonded FRP Technology (obtained with [5]).

These charts show an overview of the construction tasks, task dependencies, as well as the critical path (shown in dark). These charts are provided to illustrate the general construction process of the conventional and FRP methods. Also, Figures I.D.3.1, and I.D.3.2 illustrate one key advantage of the FRP technology: traffic maintenance (bridge closure) is dependent on cure time for the conventional repair and is a critical path item. However, traffic maintenance (lane closure) is not a critical path for the surface-bonded FRP technology. During the application of the FRP technology one lane of the bridge can remain open resulting in less traffic delays. Also, this example bridge is opened 8 days sooner. This matter is discussed later in Section I-D.3.4.6.

I-D.3.2 Overview of Design Assumptions

Surface-Bonded FRP Technology:

From designs and drawings provided by PennDOT District-3, an approximate geometry for a standard concrete T-Beam was used to estimate the amount of FRP to be used per linear foot of T-Beam to be repaired (along the beam). From this approximate geometry, a standard T-Beam section was designed. Figure I.D.3.3 is the standard T-Beam section for the design assumptions.

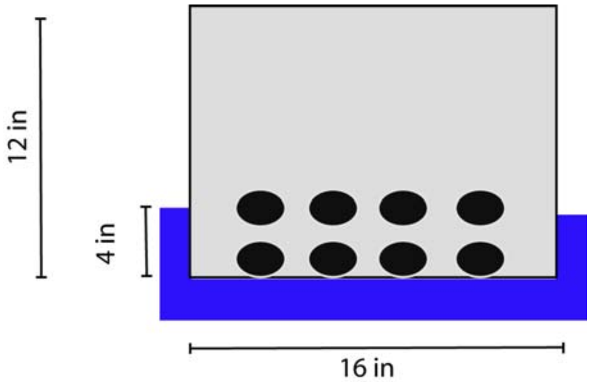


Fig. I.D.3.3. Standard T-Beam Section for FRP Design Assumptions.

Conventional Technique:

For repair using the conventional technique, an approximate geometry for a standard concrete T-Beam (insight provided by PennDOT District-3) was used to estimate the amount of concrete to be used per linear foot of T-Beam to be repaired (along the beam). From this approximate geometry, a standard T-Beam section was designed. Figure I.D.3.4 is the standard T-Beam section for the design assumptions.

I-D.3.3: Level 1 Repair (Bridge # 41-0118-0230-0644)

Bridge #41-0118-0230-0644 (Figure I.D.3.5) was selected as a Level-1 repair because this bridge shows extensive overall damage. This bridge has the following characteristics: Total Span = 80 ft (two spans); Year Built = 1943; Number of beams = 7; ADT/ADTT = 2600/300; Overall Classification = 71.7 - Class 1; Visual Rating = 2 (moderately favorable for repair). Though this bridge was originally rated “2” in the visual category, additional photographical evidence justifies high favorability for repair of this structure.

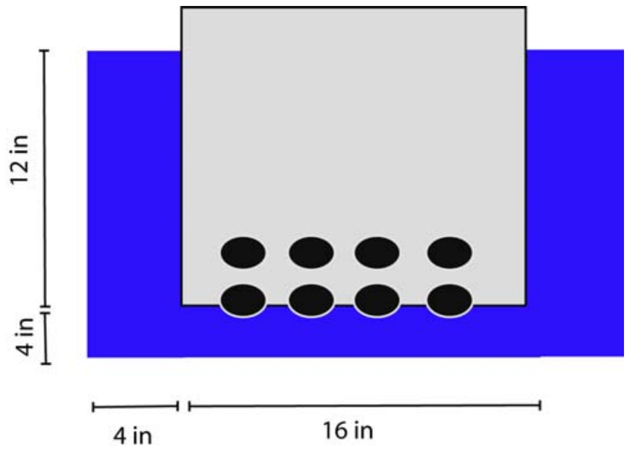


Fig. I.D.3.4. Standard T-Beam Section for Conventional Design Assumptions.

Fig. I.D.3.5. Bridge #41-0118-0230-0644

Because of the type and extent of damage, repair using the conventional method may not be possible; hence the repair will be compared to replacement cost. The total span length of this bridge is 80 feet and there are 7 beams. For complete repair, about 560 feet of beam will be repaired using the FRP technology. Though not considered in the analysis, the damaged center pier may also be a candidate for repair using FRP technology (Figure

I.D.3.6). For this example repair, only one design scenario will be considered and the "first-cost" value will be used (for information on "first cost" see Section I-D.3.4.1). This design and repair will require a significant endeavor and will likely be contracted out completely.

Fig. I.D.3.6. Photo Showing Damage to the Center Pier of Example Bridge #1.

I-D.3.3.1: Design and Cost Assumptions for FRP Repair for Example 1

<u>Design Assumptions for FRP Repair:</u>

From designs and drawings provided by PennDOT District-3, an approximate geometry for a standard concrete T-Beam was used to estimate the amount of FRP to be used per linear foot of T-Beam to be repaired (along the beam). From this approximate geometry, a standard T-Beam section was designed (for Details, See Section I-D.3.2).

Assuming two layers of FRP, and the standard design assumptions 2 x (16 in + 2x4 in), a total of 48-in or 4 ft of FRP per unit length of beam is required. For this example, 560 feet of beam is to be repaired using FRP technology. Multiplying 560 feet of beam by 4 ft per unit thickness yields a required cross-sectional area of FRP to be 2240 ft^2.

Cost Assumptions for FRP repair:

Using information provided by Mr. Steve Tysl, a consultant from Proto Composites, the cost estimate for a single repair of this bridge using the FRP technology is about $40,000 "first cost". Details of this estimate are as follows.

Using the design assumptions, 2240 square feet of FRP will be required for repairs. FRP is about $6.50-8.00 per square foot. For this estimate the cost of FRP is assumed to be $7 per square foot. Multiplying the amount of material by the material cost yields $15,700. To account for the cost of patching mortar and labor (for most projects this includes mobilization, small tools, and incidentals) as well as designing and contracting fees, an increase of 2.5 times will be assumed. By multiplying 2.5 by $15,700, we estimate about $40,000 for the repair cost. This cost estimate does not include cost of lane closure.

A parallel study from the University of Missouri-Rolla (URM) is in the process of developing a similar cost analysis. Due to the confidentiality of pricing details shared by manufacturers, no direct information can officially be provided. However, information from the ongoing study at UMR supports the overall cost estimate given in this section.

It should be noted that none of the costs described can be generalized or estimated with a great level of precision. Because the application of the FRP technologies to bridges is in its introductory phase of the product life cycle, practices and related costs vary significantly. Also, bidding prices may vary significantly.

I-D.3.3.2: Cost Assumptions for Replacement

Replacement of this bridge would cost an estimated $1.1 million "first cost". Details of this cost are as follows.

Mr. Todd Hardy from PennDOT District-3 quoted the cost of a new bridge to be about $250-300 per square foot of bridge. In order to account for removal of the existing structure, an assumed cost of $275 per square foot will be used. This new bridge will have a span length of 80 feet and will be 50 feet wide. Multiplying 80 feet by 50 feet, the total bridge area is 4000 square feet. Multiplying this by $275 per square foot yields an estimate of about $1.1 million.

I-D.3.3.3 Comparison of Costs for Example 1

Savings of FRP repair over replacement based on first cost is significant. Though the life-span of FRP repair is generally less than the life-span of a new structure, this repair would be a good example to illustrate the implementation of the FRP technology.

I-D.3.4: Level 2 Repair (Bridge # 59-0045-0310-2011)

Bridge #59-0045-0310-2011 (Figure I.D.3.7) has the following overall characteristics: Span = 30 ft, width = 43.0 ft; Year built = 1938; Number of beams = 9 (exterior beams = 20-in x 15.5-in, interior beams = 15-in x 15.25-in); Slab thickness = 8.5-in; ADT/ADTT = 10,300/650; Overall Classification = 84.8-Class 1;Visual Rating = 3 (most favorable for repair). See Figure I.D.3.8 for a cross-sectional diagram of this bridge. This example bridge has 3 damaged beams. The extent of damage on one beam is the entire length (30 feet). Another beam is damaged in two places for about 4 feet and about 5 feet in lengths. The third beam is damaged for about 15 feet. Thus, the total length of beam to be repaired is about 54 feet. The damage lengths are estimates based on photos and actual inspection of the bridge. The application of the composites will likely require more length of beam. Due to assumed lengths and unknown underlying conditions, the repair lengths will be increased by 10 percent, or a total repair length of about 60 feet.

I-D.3.4.1: Cost-Benefit Comparison for Example 2

According to [6], "First cost is the simplest and most frequently used analysis, and includes only the initial capital costs. It does not attempt to place a dollar value on future expenditures. This format of analysis is suitable for comparing alternatives with equivalent life expectancy, performance and maintenance. If significant differences are expected in one of these factors, first cost analysis will not give a true comparison of cost effectiveness of the various alternatives. Life cycle cost analysis is similar to first cost analysis except that future costs are also considered. Typically, future costs include maintenance, future rehabilitation expenditures, and probable replacement costs. In life cycle cost analysis, future costs must be discounted to present worth before they are combined with present (immediate) costs.” When evaluating first-cost in relation to traditional methods, bonded FRP technology can be more expensive, but the key advantages of FRP repair are often overlooked in relation to their high material and manufacturing costs. FRP repairs can be cost-effective in the long run if their use leads to fewer re-repairs [1]. A direct comparison of the unit price basis may not be appropriate, but rather an overall project and life-cycle cost. A present-worth projection analysis will be conducted on the example bridge with estimates for material cost, installation, traffic detour cost and future maintenance. This analysis will be based on two scenarios of bridge maintenance, as discussed in the next section.

Fig. I.D.3.7. Bridge #59-0045-0310-2011.

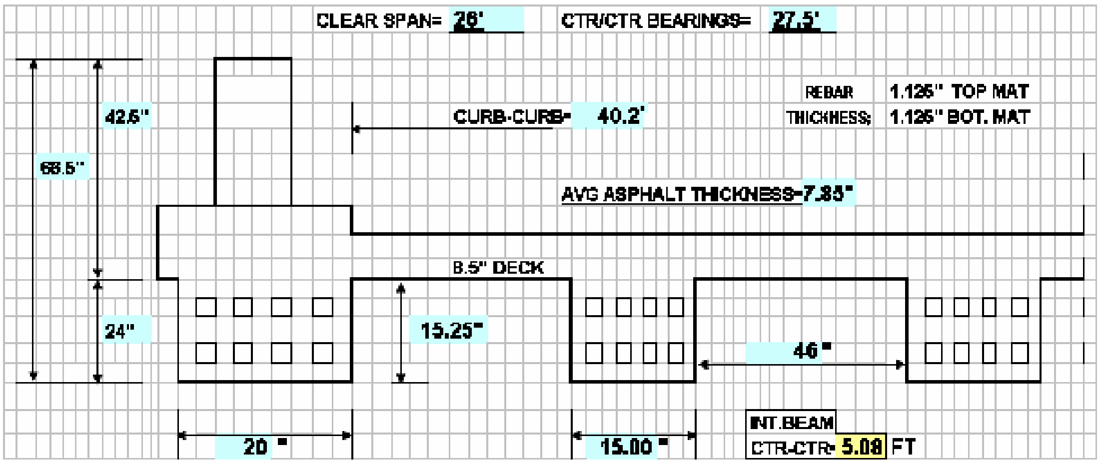


Fig. I.D.3.8 Cross-Sectional Diagram of Bridge #59-0045-0310-2011, Provided by PennDOT District-3.

I-D.3.4.2: Repair Scenarios of Example Bridge for Example 2

The considered scenarios will analyze the future-worth of both the conventional method and the surface-bonded FRP technology. The scenarios are as follows:

Scenario 1 (Design Life = 30 years)	
FRP Repair	Conventional Repair
30 year design life	15 year design life, repair twice
Replace or re-repair after 30 years	Replace or re-repair after 30 years

Scenario 2 (Design Life = 30 years)	
FRP Repair	Conventional Repair
30 year design life	20 year design life
Replace or re-repair after 30 years	Allow bridge to degrade, post bridge
	Replace or re-repair after 30 years

It should be noted that the 30 year design life for FRP technology is a conservative estimate. Sources have quoted a design life of FRP repairs up to 50 years.

I-D.3.4.3 Design and Cost Estimate for FRP Technology and Conventional Method for Example 2

For Bridge # 59-0045-0310-2011, 60 feet of repair will be assumed. The cost of FRP repair and the conventional repair are both about $3400 "first cost". Details of this calculation are as presented next.

Design Assumptions for FRP repair:

From designs and drawings provided by PennDOT District-3, an approximate geometry for a standard concrete T-Beam (Section I-D.3.2) was used to estimate the amount of FRP to be used per linear foot of T-Beam to be repaired (along the beam). From this approximate geometry, a standard T-Beam section was designed.

Assuming two layers of FRP, and the standard design assumptions 2 x (16 in + 2x4 in), a total of 48-in or 4 ft of FRP per unit length of beam is required. For this example, 60 feet of beam is to be repaired using FRP technology. Multiplying 60 feet of beam by 4 ft per unit length yields a required cross-sectional area of FRP to be 240 ft^2.

Cost Assumptions for FRP repair:

Using information provided by Mr. Steve Tysl, a consultant from Proto Composites, the cost estimate for a single repair of this example using the FRP technology is about $3400 "first cost". Details of this estimate are given next.

Using the design assumptions (Section I-D.3.2), 240 square feet of FRP will be required for repairs. FRP is about $6.50-8.00 per square foot. For this estimate the cost of FRP is assumed to be $7 per square foot. Multiplying the amount of material by the unit cost yields a total of $1680. To account for the cost of patching mortar and labor (for most projects this includes mobilization, small tools, and incidentals), an increase of 2 times will be assumed. Multiplying 1680 by two, we obtain $3360 or about $3400 for the repair. This cost estimate does not include cost of lane closure, but does include labor and other costs associated with a contracted project.

A parallel study from the University of Missouri-Rolla (URM) is in the process of developing a similar cost analysis. Due to the confidentiality of pricing details shared by manufacturers, no direct information can officially be provided. However, information from the ongoing study at UMR supports the overall cost estimate given in this section.

It should be noted that none of the costs described can be generalized or estimated with a great level of precision. Because the application of the FRP technologies to bridges is in its introductory phase of the product life cycle, practices and related costs vary significantly.

Design Assumptions for Conventional repair:
With the assumed dimension from Section I-D.3.2, a total cross-sectional area of 160 in^2 or 1.11 ft^2 per unit length of beam is required. A length of 60 ft of T-beam is to be repaired for this example. Multiplying 60 ft by 1.11 ft^2 yields a total volume of concrete of about 70 ft^3 for the repair.

Cost Assumptions for Conventional Repair:
Using information provide by PennDOT District-3, the cost estimate for a single repair of the example bridge is about $3400 "first cost". Details of this estimate are as follows.

Using the design assumptions, about 70 ft^3 of concrete will be required for repairs. Mr. Todd Hardy from District-3 has estimated the conventional repairs to cost $22-30 per ft^3 of concrete in place for materials and construction, and an additional $25 per ft^3 for labor. Assuming a combined cost of $50 per ft^3 of concrete and multiplying the amount of material by the unit cost yields an estimate of about $3400. This does not include cost of detours or traffic control.

I-D.3.4.4 Present Worth of Scenario 1 for Example 2

In order to compare initial and life-cycle costs for the conventional and FRP techniques, a present worth analysis will be conducted for Scenario 1, which includes a re-repair at n = 15 years. An interest rate of 5% is used, with a future cost of $3400 for the re-repairs. The present worth formula [7] is as follows:

$$P = F(1+i)^{-n} \qquad \text{———————} \qquad (1)$$

Where:
P = Present Worst (dollars)
i = interest rate (decimal)
n = number of periods (years)

Using the above equation (1), the present worth of a future repair ($3400) is $1635 or about $1600. Adding this cost to the initial cost, the present worth cost of Scenario 1 is $5000 ($1600 + $3400) for the conventional method. A present worth analysis does not apply to Scenario 2 because it does not include a future re-repair.

I-D.3.4.5: Cost Comparison for Scenarios 1 and 2 for Example 2

From present worth calculations, design assumptions, and cost assumptions, a comparison of costs for Scenarios 1 and 2 is as follows: For both Scenarios, the cost of the FRP repair is $3400 for an assumed 30 year design life. For Scenario 1, the present worth cost of the conventional repair is $5000. For Scenario 2, the cost of the repair using the conventional technique is $3400. Figure I.D.3.9 shows a cost comparison between methods for both Scenarios 1 and 2. The estimates for the FRP technology includes contracting costs (profit, overhead, etc.), however once the FRP technology can be implemented by district forces, costs may decrease significantly.

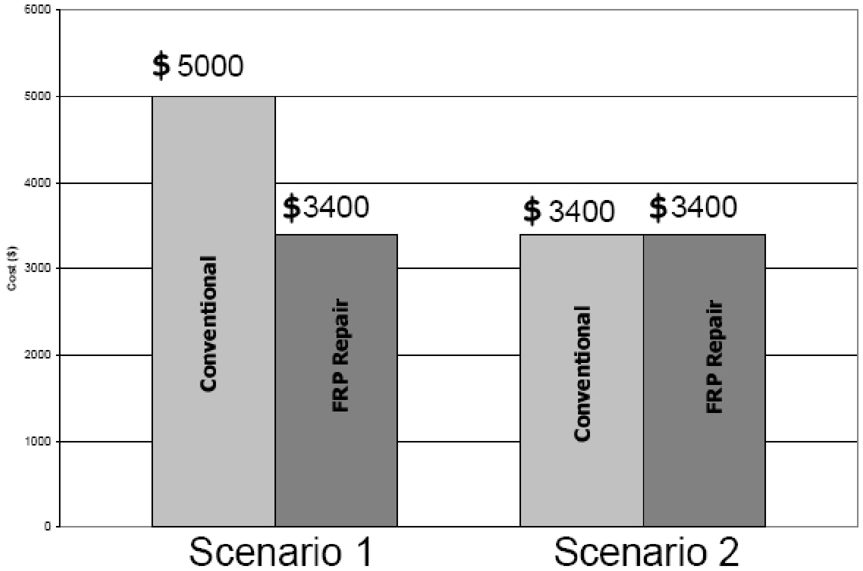


Fig. I.D.3.9. Cost Comparisons Between Conventional and FRP Methods for Example 2, Scenarios 1 and 2.

I-D.3.4.6: Detour-Driving Mileage Costs for Example 2

A key advantage of the FRP technology is that, for a typical Level-2 repair, this method allows opening the bridge about 9 days sooner than conventional repair (see Section I-D.3.1). Also, during construction using the FRP technology, one lane may remain open to traffic. Because of high traffic volumes on this bridge, a detour-driving cost analysis was performed. Using information provided by PennDOT District-3, in combination with the software package Microsoft Streets and Trips [8], and maps provided by PennDOT District-3, a detour scenario was created. Figures I.D.3.10 and I-D.3.11 show the original route and the shortest detour.

Microsoft Streets and Trips [8] is a general driving and distance mapping program, which was used to estimate driving distances for the detour scenario presented in this section.

The un-detoured length of highway is 600 yards or 0.35 miles. The proposed detour is 1.5 miles, with a difference of 1.15 miles per vehicle. From information provided by PennDOT, the ADT plus ADTT for this bridge is 10,950 (10,300 ADT and 650 ADTT), resulting in a total of about 12,600 vehicle miles per day. Using the typical IRS rate of $0.32 per mile, this cost is $4000 per day.

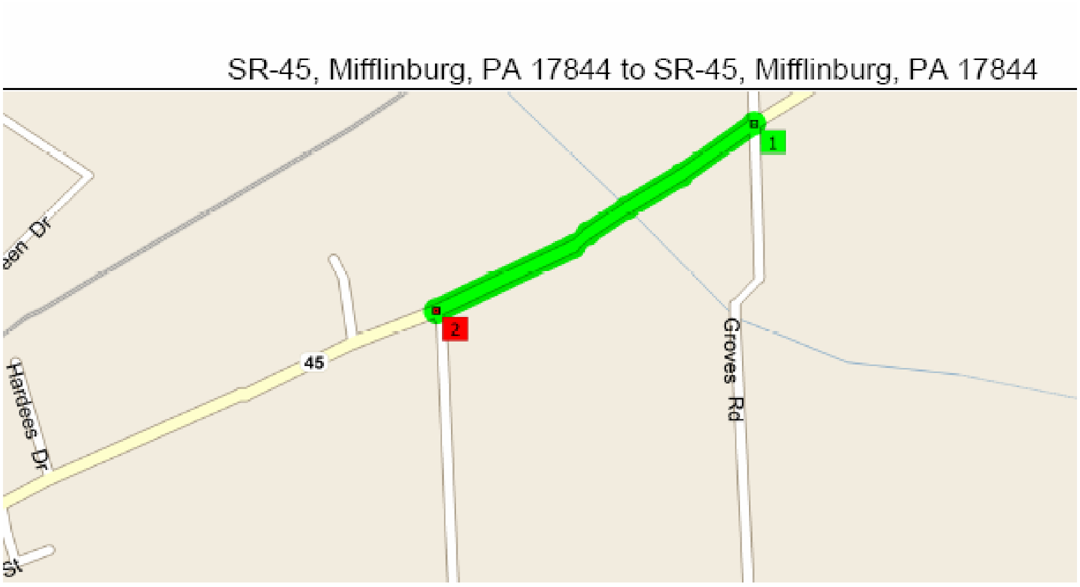


Fig. I.D.3.10. Open Traffic Plans for Example Bridge 2 (distance = 600 yards) [8].

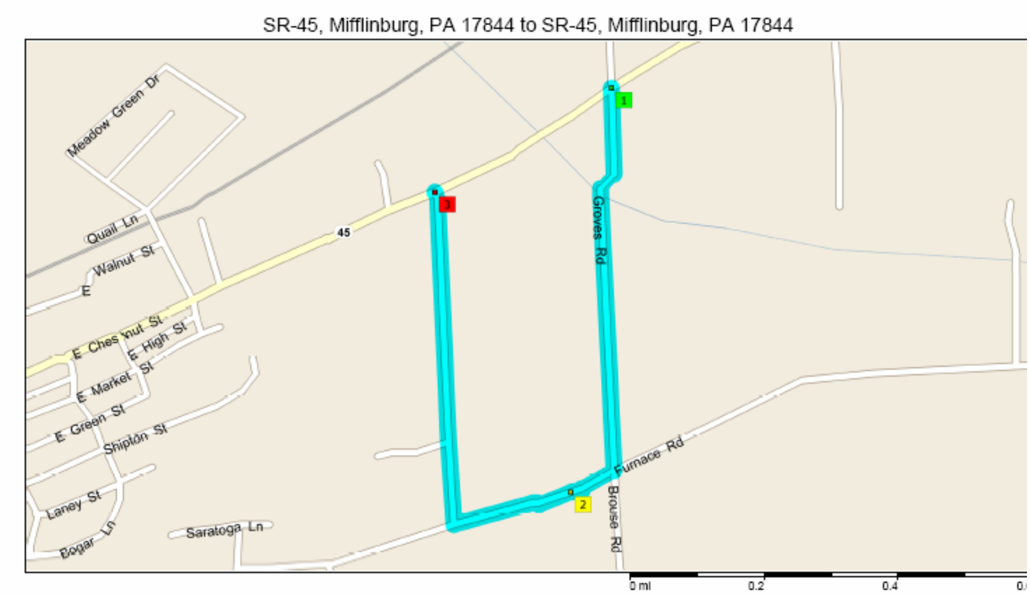


Fig. I.D.3.11. Proposed Detour Plan for Example Bridge 2 (distance = 1.5 miles) [8].

I-D.3.5: Level 3 Repair (Bridge # 41-0118-0020-1109)

From the inventory, Bridge # 41-0118-0020-1109, which shows extensive but localized damage (one beam only), was selected as a Level-3 repair project (Fig. I-D.3.12). This bridge has the following characteristics: Total Span = 31 ft; Year Built = 1930; Number of beams = 7; ADT/ADTT = 5600/410; Overall Classification = 79.8 - Class 1; Visual Rating = 3 (high desire for repair).

This bridge is relatively small and has experienced localized damage typical of the PennDOT inventory (extensive localized damage on exterior beam(s) only.) This bridge is a good candidate for repair using a trained, in-house PennDOT crew. Once trained (such as by participating in the implementation of repairs for Examples 1 and/or 2), PennDOT District-3 "in-house" forces can repair this and many other typical bridges in the inventory using FRP technology. A generalized "first-cost" estimate is provided in this section using design and cost assumptions from Section I-D.3.2 and I-D.3.4.3 respectively.

Fig. I.D.3.12. Bridge #41-0118-0020-1109.

I-D.3.5.1: Design and Cost Estimate for FRP Technology and Conventional Method for Example 3

For Bridge # 41-0118-0020-1109, 30 feet of repair will be assumed. The cost of FRP repair is about $1700, and the cost of conventional repair is about $1800. Details of these calculations are presented next.

<u>Design Assumptions for Surface-Bonded FRP Technology:</u>

Assuming two layers of FRP, and the standard design assumptions (Section I-D.3.2), 2 x (16 in + 2x4 in), a total of 48-in or 4 ft of FRP per unit length of beam is required. For this example, 30 feet of beam is to be repaired using FRP technology. Multiplying 30 feet of beam by 4 ft per unit length yields a required cross-sectional area of FRP to be 120 ft^2.

<u>Cost Assumptions for FRP repair:</u>

Using information provided by Mr. Steve Tysl, a consultant from Proto Composites, the cost estimate for a single repair of the example bridge using the FRP technology is about $1700 “first cost”. Details of this estimate are given next.

Using the design assumptions (Section I-D.3.2), 120 square feet of FRP will be required for repairs. FRP is about $6.50-8.00 per square foot. For this estimate the cost of FRP is assumed to be $7 per square foot. Multiplying the amount of material by the unit cost yields a total of $840. To account for the cost of patching mortar and labor (for most projects this includes mobilization, small tools, and incidentals), an increase of 2 times will be assumed. Multiplying $840 by two, yields $1680 or about $1700 for the repair. This cost estimate does not include cost of lane closure, but does include labor and other costs associated with a contracted project.

A parallel study from the University of Missouri-Rolla (URM) is in the process of developing a similar cost analysis. Due to the confidentiality of pricing details shared by manufacturers, no direct information can officially be provided. However, information from the ongoing study at UMR supports the overall cost estimate given in this section.

It should be noted that none of the costs described can be generalized or estimated with a great level of precision. Because the application of the FRP technologies to bridges is in its introductory phase of the product life cycle, practices and related costs vary significantly.

<u>Design Assumptions for Conventional Technique:</u>

Using assumed dimensions (Section I-D.3.2), a total cross-sectional area of 160 in^2 or 1.11 ft^2 per unit length of beam is required. For the sample bridge, 30 ft of T-beam is to be repaired. Multiplying 30 ft by 1.11 ft^2 yields a total volume of concrete of about 35 ft^3 for the repaired.

<u>Cost Assumptions for Conventional Repair:</u>
Using information provide by PennDOT District-3, the cost estimate for a single repair of this example is about $1800 "first cost". Details of this estimate are as follows.

Using the design assumptions, about 35 ft^3 of concrete will be required for repairs. Mr. Todd Hardy from District-3 has estimated the conventional repairs to cost $22-30 per ft^3 of concrete in place for materials and construction, and an additional $25 per ft^3 for labor. Assuming a combined cost of $50 per ft^3 of concrete and multiplying the amount of material by the unit cost yields a total of about $1800. This does not include cost of detours or traffic control.

I-D.3.6: Learning Curve Approach

There are arguments that initial costs of new technologies decrease with time, as their use become more extensive and accepted. According to Ref. [9], "Bridges that use FRP are expected to have higher initial costs than traditional reinforced concrete bridges, due to high cost of fiber and resins. However, this initial cost will decrease as more bridges are repaired according to the Learning Curve theory. The Learning Curve theory predicts that, as experience builds up, the cost will decrease in an exponential manner. Typical costs start high, but drop steeply when methods and materials become more cost effective as the product matures [See Figure I.D.3.13]. Over time, the large inefficiencies are removed from the process and the costs stabilize."

In this task, we presented a comparative overview of repair approaches by PennDOT's District-3 conventional method and FRP technology. Also, an example was provided comparing costs between FRP repair and replacement of the bridge. Example bridges from Missouri with similar concrete T-Beam designs were presented first to provide an insight into the construction process of FRP repair technology. Then, three example bridges from the PennDOT District-3 inventory were chosen to illustrate the differences in processes and costs.

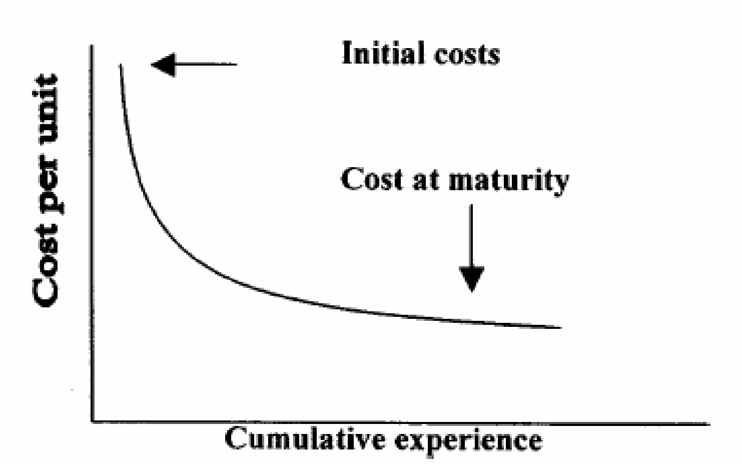


Fig. I.D.3.13. Typical Learning Curve [9]

Sub-Task I-D.4: Analysis and Conclusions

It was shown that, for a Level-1 repair the cost of FRP technology is quite inexpensive in comparison to the cost of bridge replacement. The Level-1 repair presented in Example 1 (Section I-D.3.3) would provide familiarity and training experience for PennDOT, to subsequently allow them to implement FRP repair using District forces. Based on experience from this repair, PennDOT District-3 could begin using the FRP technology for a wide scale repair of deteriorating T-Beam structures, such as the suggested Example 3, a Level-3 repair effort (Section I-D.3.5).

For a Level-2 repair (Example 2), two repair scenarios comparing the conventional method and FRP technique were provided to illustrate the "present-worth" and "life-cycle" methodology. For both scenarios, it was shown that the cost of FRP technology is competitive with the cost of the traditional method, and when "life-cycle" repairs were included, the FRP technology is less expensive. Also, it is likely that once the FRP technology becomes more widely accepted and used, both material cost and experience in installation will improve (Learning Curve theory).

Setting aside cost differentials, the FRP technology has many other advantages over the conventional repair method. The most pronounced advantage is speed of application. The FRP technology may allow for the bridge to remain open during repairs. Also, the installation and curing time for the FRP technology is quite short, in comparison to the more complex forming and longer curing of concrete encasing. Thus the rapid application of FRP repair can lead to reductions in detour costs and inconveniences. Also, FRP material installation is quite simple; the material is light and can be installed using hand tools. There is a high likelihood that, once trained, the technology can be applied by state employees. Though gains in structural strength are generally not of concern for the PennDOT inventory, bridges repaired using the FRP technology will experience increases in strength and therefore have higher capacities. Since the costs of the FRP technology included contracting fees (profit, overhead, etc), the application of the FRP technology may become significantly less expensive once this repair is performed by PennDOT "in-house" forces.

It has been shown that the FRP technology is being applied and working satisfactorily in several state DOTs, such as Missouri. Repairs using this technique are easier and faster to implement. Also, these repairs tend to be more durable than those using conventional techniques, allowing for less frequent re-repairs.

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Section 5 - Task I-F: Implementation Strategy

Upon a decision by the PennDOT Project Advisory Panel to proceed with the implementation phase of this project, the research team, if authorized by PennDOT, will submit a detailed proposal for Phase II. This section provides a tentative implementation strategy for Phase II, which includes: (1) identification of candidate bridge, (2) field assessment of the structure, (3) evaluation of in-situ material properties from field samples, (4) structural analysis for existing conditions, (5) design of FRP repair and other work, (6) repair implementation and evaluations, (7) testing of repaired structure, (8) supporting lab-scale studies, and (9) development of draft specifications and standard drawings.

I-F.1 Identification of Candidate Bridge

The recommendations for identifying a candidate bridge for repair were given in Section I-C.3, based on age, span, ADT, ADTT, and visual rating. Three candidate bridges were identified from PennDOT District-3 bridge inventory for major, moderate, and minor repair. The field implementation strategy described in this section is for Level-1 candidate bridges, concerned with major repair by contracting the work out.

I-F.2 Field Assessment of the Structure

- Inspection: Observing and recording overall damage conditions including concrete spalling, rebar corrosion, and general cracking and damage; obtaining the Bridge Inspection Report and the Load Rating Sheets of the candidate bridge from the PennDOT inspection unit; reviewing key information of the bridge including span, width, and rating.
- Existing material conditions: Measuring remaining cross-section of rebar, cutting a sample from an exposed rebar for strength testing; measuring remaining concrete dimensions and obtaining cored samples from the deck for compression strength testing.
- Quality of concrete substrate: Performing tension test.

I-F.3 Evaluation of In-situ Material Properties from Field Samples

- Determining average yield strength of rebar by testing specimens cut from an exposed bar at an appropriate location; conducting the test according to ASTM A615 and ASTM A955; assuming a steel grade for design based on the test results.
- Determining concrete compression strength by testing cored samples obtained from the deck, in compliance with ASTM C39/C39M-1 and ASTM C42/C42M-99; determining concrete cover and size of longitudinal and transverse steel bars in the deck from the concrete cores.

I-F.4 Structural Analysis for Existing Conditions

- Analyzing as-built drawings, original design sheets or performing further field surveys to ascertain the initial data for the bridge.
- Calculating ultimate values of bending moments and shear forces according to AASHTO, 2002.
- Defining design truck, load lanes and design lanes as per AASHTO, 2002.
- Load transfer and slab analysis to determine bending moments and shear forces.
- Girder analysis to determine bending moments and shear forces.
- Bent analysis to determine bending moments and shear forces.
- Abutment analysis according to ACI 318-02 to determine axial load strength.
- Field testing to determine strain distributions under different service load cases before strengthening, using attached strain gages and LVDTs, or survey equipment as an alternative.

I-F.5 Design of FRP Repair and Other Work

Strengthening design will be carried out according to the principles of ACI 440.2R-02 and AASHTO.

- Calculating material properties of FRP laminates considering long-term exposure to environmental conditions.
- Slab design: Slab flexural strengthening of positive moment region, negative moment check, and shear check.

- Girder design: Positive moment strengthening, negative moment check, and shear strengthening.
- Bent design: Positive moment strengthening, negative moment check, shear capacity check, and pier check.
- Load rating: Determining the rating of slab, girders, bents, and piers based on AASHTO specifications.

I-F.6 Repair Implementation and Evaluations

- Concrete repair: Restoring the concrete cross section including chipping and removing damaged concrete, replacing steel reinforcing when necessary, replacing concrete, and repairing cracks, according to ACI 546R-96 and ICRI No. 03730.
- Surface preparation: Performing surface grinding, chamfering corners, crack injection, surface profiling, and surface cleaning according to ACI 224.1R, ACI 224R-01, and NCHRP Report 514.
- Installation of FRP system: Three types of FRP systems may be used; specific procedures for installing them may vary slightly.
  - *Manual lay-up FRP system*: mixing of resin components; primer and putty; saturant; application of FRP sheet and saturant; overlapping; alignment of FRP materials; anchoring of FRP sheets; stressing applications; protective coating and finishing.
  - *Near surface mounted FRP system*: cutting grooves in concrete; application of embedding paste; placing FRP reinforcement; protective coating and finishing.
  - *Precured FRP system*: application of adhesive; placement of precured system; anchoring of precured system; grouting of precured system; stressing applications; curing; protective coating and finishing.
- Acceptance testing and inspection: Soundings of the strengthened areas to check for voids, bubbles, and delaminations; a surface adherence pull-off test, surface adherence shear test, or surface adherence torque test (modified ASTM D 4541-93) to measure bond strength; inspection of materials, fiber orientation, cure of

resin by testing panels or resin-cup samples using ASTM D 3418, cured thickness.

- Rework and repairs: Defective or damaged sections of applied composite systems will be repaired or replaced in compliance with the requirements of specification if deemed necessary by PennDOT Engineer; the rework and repairs may include repair of protective coating, epoxy injection of small defects, patching of minor damages, and replacement of large defects.

I-F.7 Testing of Repaired Structure

An in-situ conventional load testing will be conducted on the retrofitted structure to determine if the FRP system is performing appropriately and has increased the flexural capacity of the bridge. A non-destructive test method - the static diagnostic load method can be used for the service load testing of the bridge. Strain gages and LVDTs will be attached to measure the structural behavior of the bridge. Strain gages will also be placed on the FRP strips to measure the strains and to confirm the composite action of the strengthened components. The performance of the structure prior to and after the strengthening will be determined by comparing the normalized experimental results prior and after strengthening.

A dynamic test will be conducted on the strengthened bridge in order to determine the impact factor. In addition, a Finite Element analysis model will be developed in order to interpret the experimental data prior to and after the strengthening.

I-F.8 Supporting Lab-scale Studies

- Auxiliary tests on witness panels will be carried out following ASTM D3039; on at least five witness panels for each type of system, to measure strength, elastic modulus, and ultimate strain. The FRP system will be unacceptable if the average tensile strength and the lowest tensile strength are more than 5% and 10% below that specified in the contract document, respectively (NCHRP Report 514).
- Chemical evaluations of concrete core samples including the chloride diffusion (ASTM C 1152), pH of concrete and corrosion potential of the reinforcement.

- Visual and petrography examination of the concrete to evaluate the in-situ alkali-aggregate reactions if any (ASTM C 856).
- SEM/EDAX and powder X-ray diffraction of samples to study degradation of concrete at micro-structural level.
- Produce scaled T-beam samples with similar properties as the in-situ structures.
- Strengthens the T-beam samples with the FRP system using same materials and methods used for the retrofitted bridge.
- Test the control and strengthened beams under static and fatigue loading to investigate the effect of the strengthening on the ductility and failure modes of the beams.
- Accelerated aging of coupon samples to simulate field conditions, such as moisture diffusion, freeze-thaw cycling in salt solution, and wet-dry cycling in alkaline solution.
- Test the control and conditioned samples for pull-out interface strength, and also opening-mode interface fracture, to indirectly evaluate the as-repaired and future FRP-concrete interface strength and integrity. Single Contoured-Cantilever Beam and Single-lap Shear specimens will be used to mimic the actual failure of FRP-concrete interface under Mode-I and Mode-II loading, respectively.
- Predict the response of the test samples under static and fatigue loads using the Cohesive Zone Model with interface strength and fracture parameters obtained from the testing of coupon samples above.
- Develop S-N curves for the different aging conditions considered, and obtain master-curves using Arrhenius model to predict long-term performance of FRP-concrete interface.

I-F.9 Development of Draft Specifications

Ultimately, the culmination of Phase II efforts will be the development of a comprehensive Load and Resistance Factor Design (LRFD) specifications, standardized drawings, and construction specifications for the use of FRP repair and rehabilitation procedures developed in this project.

Conclusions and Recommendations

Based on the research study conducted the following can be concluded:

1. FRP composites are being used in new bridge construction and rehabilitation. The most promising applications of FRP composites in civil engineering are in structural rehabilitation. Recent FRP rehabilitation projects have varied characteristics and environmental conditions, in which beams rank highest for all the application situations. The most common overall pre-repair conditions for the repaired structures are severe deterioration and loss of concrete section. The overall satisfaction was very high with FRP technology; and almost all states recommended the technology for other repair/retrofit projects.
2. The behavior of concrete members strengthened or retrofitted with FRP systems is highly dependent on a sound concrete substrate, and also proper preparation and profiling of the concrete surface.
3. There are three common FRP system forms suitable for the strengthening of structural members. These are wet layup systems, prepreg systems, and precured systems. Wet lay-up and precured FRP systems may be prestressed to improve their performance.
4. Composites can be applied in three ways: adhesive bonding, wet lay-up, and resin infusion. Adhesive bonding and wet lay-up are the most widely used.
5. Tension face plates, side plates, and combination plates are used to improve the flexural capacity, the shear capacity, and both shear and flexural capacities, respectively. These plates can be bolted, adhesively bonded, or attached to the concrete surface mechanically using fasteners.
6. Shear strengthening includes bonding to the sides of the beam, U-jacketing around the bottom, and total wrapping of the beam.
7. Anchor bolts, L-shaped plates, U-anchors, near surface mounted rods, and mechanical clamps have been used to prevent debonding from happening.
8. A coordinated effort among materials and technology supplier, in-house design/materials division, research institute/university, and applications contractor,

provided effective implementation protocols in most projects. Generally, qualified research institute/university consultant and materials supplier are available.

9. Externally bonded FRP laminates are very effective to increase the ultimate load capacities of reinforced concrete beams. The more layers of FRP are used, the higher flexural strength and shear capacity are achieved. Prestressing FRP laminates represent a more efficient use of the technology. A significant loss in beam ductility can occur when FRP materials are used for flexural strengthening. A triaxially braided ductile fabric can increase ductility index than commonly used carbon fiber sheets.
10. There is not sufficient information to predict long-term performance with certainty, but predictions can be made from the results of accelerated tests using different statistical or analytical models such as Arrhenius model. There is not enough data to verify long-term properties of the bond interfaces; however, if the right type of material is used, and if the strengthening work is carried out carefully, 30 years of use can be guaranteed. Unacceptable reductions in mechanical properties can occur if resins with inadequate moisture absorption characteristics are employed. Freeze-thaw and wet-dry cycling will induce significant deterioration of the bond.
11. First-costs of FRP technology are more expensive in relation to traditional methods. When installation is included in the cost comparison, FRPs can compete with conventional materials. If the comparisons include reduced overall life cycle costs, in light of the crippling need for large maintenance budgets, FRP can have a significant advantage.
12. In relation to conventional repair methods, the FRP technology was shown to be significantly more cost effective for Level 1 repair, particularly in relation to total bridge replacement, and either less or equally costly for Levels 2 and 3, depending on the scenarios considered.
13. The FRP technology does have disadvantages and limitations such as higher initial materials cost, lack of comprehensive standards and design guidelines at present, increases in strength may not more than 50% because of ductility, unable to prevent on-going corrosion, limited effectiveness for punching shear responses in slabs or

footings, limited effectiveness if significant portion of core concrete is lost, and limited effectiveness if members are too deteriorated.

In summary, this project offered District-3 a timely opportunity to develop a comprehensive practical study of the FRP technology. District-3 would significantly benefit from implementation of FRP technology for repair of concrete T-beam bridges, with potential application to over 50% of the 128 bridges considered in the study. Moreover, the guidelines developed through a District-3 field demonstration project can serve PennDOT in general for future applications of FRP repair and retrofit to various types of concrete bridges.

Based on the conclusions of this study, the following can be recommended:

1. "Fiber-wrap" technology can be used to repair damaged concrete bridge components in District-3. The suitable candidate bridges can be selected based on age, span, ADT/ADTT, and visual inspection. For the effective implementation of FRP-repair in a selected bridge, a preliminary analysis including a cost-benefit analysis, and a detailed analysis of the structure should be conducted.
2. It is recommended that the field implementation phase should follow sequentially bridge projects beginning with Level 1 and followed by Levels 2 to 3. This proposed approach will serve effectively to transfer knowledge to district personnel and permit them to participate in hands-on training.
3. In the following implementation phase (Phase II), it is recommended that PennDOT District 3 designate personnel from the bridge design section to learn to design strengthening systems for bridges with FRP materials. The experiences gained will lead to further in-house bridge repair projects with FRP. Also, District personnel will develop better understanding of relevant issues that should be addressed in the development of specifications for PennDOT.
4. The durability of the FRP-concrete interface bond is perhaps the major issue of pressing concern, because the long-term adequate strength and integrity of the bond is crucial for the effectiveness of FRP-concrete technology. Additional research work is recommended to develop a structured and comprehensive approach for durability studies, including testing methods, data reduction and interpretation

techniques, durability protocols for environmental exposure, and possible long-term prediction models for service-life performance.

Section 7 - Appendices

Appendix A

PennDOT District 3-0
Investigation of Surface bonded FRP Technology for Concrete Bridge Repair and Rehabilitation

Survey Questions for Assessing Current Practice

- **Applications of Surface bonded FRP Technology**
- **Implementation Protocols**
- **Evaluations and Assessments**

Project Significance and Purpose of the Survey

PennDOT's Engineering District-3 maintains approximately 2900 bridges over nine counties, with over 300 T-beam concrete bridges built during the early- mid-1900s. These bridges are in need of repair/ retrofit and can benefit from rehabilitation techniques with externally bonded Fiber-Reinforced Polymer (FRP) composites, more appropriately termed *Surface Bonded FRP Technology*. This survey is part of a Phase-I study to evaluate the feasibility of Surface bonded FRP technology for potential applications to District-3 concrete T-beam bridges.

PennDOT has contracted the services of the research team at West Virginia University, headed by Professor Julio F. Davalos, for the analysis and possible subsequent implementation of Surface bonded FRP technology. To this end, the responses to this survey together with a review of the state-of-the-art will permit a synthesis of analysis of findings, to make recommendations to PennDOT on best practices and most effective implementation of the technology.

We ask that you take a brief moment with this survey to help us achieve the objectives of this project. The questionnaire is organized in three sections: (I) Applications, (II) Implementations, and (III) Evaluations. Please indicate your answer with a checkmark, and please provide any additional information or comments as requested. At the end of each section we ask that you provide us with brief descriptions about your opinion and experience with the technology.

Your input is invaluable for the successful completion of this project, and we appreciate your kind response.

Please fill out the following information for future contact:

Name:	Position:
Agency:	Telephone:
Street Address:	Fax:
City, State, Zip:	E-mail:

Please return this survey to:

Don Matzzie, Ph.D., P.E.
Linare Consulting
Ph: 412-343-3888
Fax: 412-343-2168
Cell: 412-576-3703
Email: dmatzzie@earthlink.net

Applications of Surface bonded FRP Technology

This section focuses on information pertaining to the applications experience your agency may have with Surface bonded FRP technology. In particular, questions related to methods used for determining applicability of the technology, any specifications used, types and purpose of applications, level of damage and specific conditions, cost effectiveness and degree of satisfaction, and finally your opinion about best conditions and limitations for technology implementation.

1. **Does your state / agency have experience with surface bonded FRP repair and/or retrofit for concrete?**
   ____ Yes; ____ Number of projects
   ____ No

   Comments:

2. **Please indicate the source of information that was used for determining the applicability of surface bonded FRP technology for repair / retrofit:**
   ____ By soliciting advice from:
      ____ Research/university collaborator
      ____ Materials / technology supplier
      ____ Consulting engineering firm
      ____ Contractor familiar with technology implementation
      ____ Other, please describe:
      ____ Based on previous experience with the technology
   ____ Based on recommendations by another DOT or project owner
   ____ Based on published information or available literature
   ____ Other; please explain:

   Comments:

3. **Does your state / agency have specifications for the application of Surface bonded FRP technologies?**
   a. Materials selection and or qualification procedures
   ____ Yes (please provide information to obtain specifications)
   ____ No

   Comments:

   b. Construction specifications and or procedures
   ____ Yes (please provide information to obtain specifications)
   ____ No

   Comments:

c. Quality control and or quality assurance procedures
____ Yes (please provide information to obtain specifications)
____ No

Comments:

d. Performance assessment procedures
____ Yes (please provide information to obtain specifications)
____ No

Comments:

4. **Please indicate the applications in which the technology was used**
___ Pier caps
___ Piers: ___ Round; ____ Square
___ Beams: ___ Rectangular / Square; ___ I-Beam; ___ Box
___ Pre-stressed; ___ Post tensioned
___ Slabs: ___ Conventional RC; ___ Waffle; ___ Post tensioned
___ Walls
___ Arches
___ Other; please explain:

Comments:

5. **What was the purpose of the application?**
____ Repair
____ Strengthen
____ Both repair and strengthen
____ Corrosion mitigation / repair
____ Seismic upgrade
____ Other; please explain:

Comments:

6. **For repaired structures, please describe overall pre-repair conditions (please consider both concrete and reinforcing steel)**
____ No apparent deterioration
____ Mild deterioration
____ Moderate deterioration
____ Severe deterioration
____ Other; please explain:

Comments:

7. **For any project(s) where surface bonded FRP repair was used, please indicate specific existing conditions where FRP technology was applied, such as:**
   ____ Loss of concrete section
   ____ Corrosion of rebar
   ____ Damage due to salt exposure
   ____ Damage due to alkali-silica exposure
   ____ Other deterioration problems; please explain:

   Comments:

8. **For any project(s) where Surface bonded FRP repair was used, please indicate overall cost assessment:**
   ____ Project cost in relation to expected service-life was:
   _____ Satisfactory or _____ Unsatisfactory
   ____ Project cost in relation to other repair methods was:
   _____ Satisfactory or _____ Unsatisfactory
   ____ Other; please explain:

   Comments:

9. **Please indicate your agency's overall satisfaction with the surface bonded FRP repair technology:**
   ____ Very satisfied
   ____ Satisfied
   ____ Indifferent
   ____ Dissatisfied
   ____ Very dissatisfied
   ____ Other; please explain:
   ____ I would ____ I would not recommend surface bonded FRP technology for other concrete repair / retrofit projects

   Comments:

10. **Please describe briefly your preferred or suggested "ideal conditions" for the effective implementation of surface bonded FRP technology for repair/retrofit of concrete bridges:**

11. **Please indicate any limitations for the effective application of this technology, considering: Practical construction issues; uncertainties in long-term performance of repair/retrofit; and cost/benefit considerations:**

Implementation Protocols for Surface bonded FRP Technology

This section focuses on information pertaining to the implementation protocols being used currently by state DOTs in collaboration with relevant industries and consultants. In particular, questions related to the management and responsibilities of the different entities engaged, including researchers, technology suppliers, consulting and/or in-house design engineers, contractors, and owners.

1. **Please indicate all entities that were involved in the project**
   ____ Research institution/university
   ____ Materials and technology supplier
   ____ Consulting engineering company
   ____ In-house design/materials division
   ____ Applications contractor different from materials supplier
   ____ Other; please explain:

   Comments:

2. **If a research institution/university was involved, please indicate its responsibilities:**
   ____ Advisory to the DOT
   ____ Coordination among all entities involved
   ____ Specification of materials, and lay-up architecture and details
   ____ Selection of technology supplier
   ____ Selection of applications contractor, if different from supplier
   ____ Supervision of field work
   ____ QA and QC application measures
   ____ Instrumentation during or after application
   ____ Inspection after application
   ____ Testing and evaluation after application
   ____ Studies for: ____Service life-span, and ____Cost effectiveness
   ____ Studies for: ____ Long-term performance, and ____ Durability
   ____ Other; please explain:

   Please provide contact information for the Research Institution / University Involved:

   Comments:

3. **If a materials and technology supplier was involved, please indicate its responsibilities:**
   ____ Field inspection and assessment of needed repair / retrofit
   ____ Materials: ____ Specification & supply, or ____ Supply only
   ____ Materials test data and physical properties/design values
   ____ Application: ____ Methods & field work, or ____ Methods only
   ____ Selection of field application sub-contractor, if different

____ Training or certification of installation personnel
____ Responsibility for all field work
____ QA and QC application measures
____ Instrumentation during or after application
____ Inspection after application
____ Testing and evaluation of structure after application
____ Studies for____ Performance and/or ____ Durability
____ Other; please explain:

Please provide contact information for the Materials and Technology Supplier involved:

Comments:

4. **If a consulting engineering company was involved, please indicate its responsibilities:**
____ Advisory to the DOT
____ Coordination among all entities involved
____ Specification of materials____ Design of fiber lay-up
____ Design details for target performance
____ Prepare contract documents
____ Administer contract / advertise bid
____ Selection of technology supplier
____ Selection of applications contractor, if different from supplier
____ Supervision of field work
____ QA and QC application measures
____ Instrumentation during or after application
____ Inspection after application
____ Testing and evaluation after application
____ Studies for: ____Service life-span, and ____Cost effectiveness
____ Studies for: ____ Long-term performance, and ____ Durability
____ Other; please explain:

Please provide contact information for the Consulting Engineering Company involved:

Comments:

5. **If a field application sub-contractor, <u>different from the materials and technology supplier,</u> was involved, please indicate its responsibilities:**
____ Field inspection and assessment of needed repair / retrofit
____ Responsibility for all field work
____ Training and certification of installation personnel
____ QA and QC application measures
____ Instrumentation during or after application
____ Inspection after application

____ Testing and evaluation after application
____ Studies for____ Performance and/or ____ Durability
____ Other; please explain:

Please provide contact information for the Field Application Sub-Contractor involved:

Comments:

6. **Please indicate your degree of satisfaction with project management, by using the following scale: (5) Very satisfied; (4) Satisfied; (3) Indifferent; (2) Dissatisfied; and (1) Very dissatisfied:**
____ Coordination of all entities involved, as described above
____ Availability of qualified research/university consultant
____ Availability of qualified materials and technology supplier
____ Availability of qualified field applications contractor / personnel
____ Availability of qualified consulting engineering company
____ Availability of qualified in-house design/materials personnel
____ Successful application of the technology
____ Successful QA/QC or evaluation methods during applications
____ Successful testing and assessment methods of the applications
____ Successful evaluations of the technology thereafter
____ Other; please explain:

Comments:

7. **Please describe briefly your preferred or suggested management approach for the effective implementation of the technology:**

Evaluations and Assessments of Surface bonded FRP Technology

This section focuses on information pertaining to evaluations and assessments of the technology. In particular, questions related to practices used for pre-repair conditions, materials and methods during construction, inspections and tests after completion, and subsequent long-term studies. Finally, we ask your overall degree of satisfaction and your opinion on best practices for evaluations and assessments.

1. **Please indicate any evaluations of surface bonded FRP repair procedures conducted by your organization for any field project(s):**
   ____ Pre-repair conditions to determine applicability of technology
   ____ Pre-approvals for materials selection and ____ qualifications ____ Proprietary methods available from different vendors
   ____ Critical QA/QC issues for field implementation
   ____ Critical test procedures ____ during / ____ after application
   ____ Design methods and performance limits
   ____ Inspection methods after construction
   ____ Field testing and ____ correlations with predictions
   ____ Cost / benefit studies for:
   ____ Initial costs; ____ Service life-span costs; and
   ____ Comparative costs with conventional methods
   ____ Other; please describe:

   Comments:

2. **Please indicate any evaluations or inspections conducted for determining acceptable <u>pre-repair substrate conditions</u>:**
   ____ Corrosion of rebar:
   ____ Problems: ____
   ____ Surface preparation:
   ____ Problems: ____
   ____ Moisture control:
   ____ Problems: ____
   ____ Temperature control:
   ____ Problems: ____
   ____ Sound concrete substrate:
   ____ Problems: ____
   ____ Chloride content of concrete and /or ____ Desalination
   ____ Problems: ____
   ____ Carbonation of concrete surface; ____ Estimated depth
   ____ Problems: ____
   ____ Other; please describe:

   Comments:

3. **Please indicate any evaluations or inspections conducted for determining acceptable repair <u>procedures during applications</u>:**

____ Crack injection for selecting materials / techniques

____ Problems: ____

____ Surface treatment of substrate concrete for proper patching

____ Problems: ____

____ Forming and patching to restore concrete section loss

____ Problems: ____

____ Surface preparation of patched and pre-existing concrete for application of resin/fiber system

____ Problems: ____

____ Resin / fiber applications and finishing

____ Problems: ____

____ Finish coat application for UV/fire/other protective coating

____ Problems: ____

____ Other; please describe:

Comments:

4. **Please indicate any evaluations or assessments conducted <u>immediately after completion of repair applications</u>:**

____ Inspection for overall quality of finished repair

____ Problems: ____

____ Inspection for delamination or "missed" spots by sounding / tapping or other technique

____ Tests on either lab-cured or field-cured coupon samples for strength and integrity of interface bond

____ Problems: ____

____ Tests on actual repaired sections for strength and integrity of interface bond;

____ By round / square dolly pull test or other

____ Problems: ____

____ Full scale proof or load test on structure or structure component

____ Problems: ____

____ Tests on either lab-cured or field-cured coupon samples for effectiveness and performance of finish coat, if any

____ Problems: ____

____ Other; please describe:

Comments:

5. **Please indicate any <u>follow-up or long-term evaluations or assessments</u> conducted:**

____ Visual inspections, photographs and documentation

____ Time intervals: ____________________________________

____ Problems: ____

____ Measurements which may include:

____ Delamination zones; ____ Humidity; ____ Stain zones
____ Damaged fibers/resin; ____ Damaged gel coat
____ Temperature; ____ Strains; ____ Displacements
____ Time intervals: ______________________________
____ Problems: __________________________________

____ Load tests:

____ Before, and/or ____ after FRP repair/retrofit application
____ Measurements taken: _________________________
____ Time intervals: ______________________________
____ Problems: __________________________________

____ Other; please describe:

Comments:

6. **Please indicate your degree of satisfaction with evaluations and assessment procedures, by using the following scale: (5) Very satisfied; (4) Satisfied; (3) Indifferent; (2) Dissatisfied; and (1) Very dissatisfied:**
____ Procedures for pre-repair conditions
____ Procedures during repair work applications
____ Procedures immediately after work completion
____ Procedures for long-term conditions
____ Other; please explain:

Comments:

7. **Please describe briefly your preferred or suggested evaluation/ assessment approach for the effective implementation of the technology:**

End of Survey: Thank you for your kind help and time!

Appendix B: Survey of State DOTs Regarding the Use of Fiber Reinforced Polymers for Repair / Retrofit of Concrete Bridge Structures

No.: *1* Interviewee: *Rick Carter* Title: *Sr. Bridge Engr Technical Specialist* Affiliation: *CalTrans Maintenance Section* Address: *Sacramento*	Phone: *916-227-8625* E-mail: *richard_r_carter@dot.ca.gov* Date Surveyed: *5/31* Notes: *referred by Sheng*

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes, agency did 3, 1 personally; refer to the one.*
2 Info sources	*Familiar contractor; published info; principles are well established in CA for bldg seismic strengthening.*
3a Materials specs?	*Project specs only; not fully developed*
3b Const procedures?	*Covered*
3c QC procedures?	*Covered*
3d Perf assess proc?	*Not officially; working on.*
4 Application?	*Damaged pre-cast girder; truck hit; surface bonded to add strength; severed strand.*
5 Purpose?	*Repair and strengthen and gain experience; proof of concept;minimal damage - less than 10%.*
6 Condition?	*Damage and lost section.*
7 Specific cond?	*Loss of concrete; truck strike -- broken prestress tendon.*
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes*
9a Overall satis?	*Satisfied*
9b Recommend?	*Yes*
10 Ideal conditions?	*Costly, but cheaper where other repair methods would disrupt traffic flow.*
11 Limitations?	*Problem because there is no good set of design specs.*

Section II Implementation Protocols

1 Entities?	*Matls supplier; applic contractor; competitive bid project; Rt 233/152 separation; specs were left open. Watson-Bowman Acme Brace; also composite column wrap.*
2 If res or univ?	*n/a*
3 If supplier?	*Supply materials; test data; methods.*
3 Matls supplier?	

3 Supplier contact?	*Watson Bowman Acme Mbrace.*
4 If engrg firm?	*n/a*
5 If applic contr?	*Was the prime; no assessment pre; all field work; training & cert; QA QC; inspection; testing & eval; there was also traffic control.*
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	*3* *na* *4* *3* *na* *2* *4* *2* *3* *2*
6 Comments:	
7 Pref. mgmt approach:	*Mix of method & performance spec; performance on matls; method spec on application. Would prefer a performance spec, ultimately.*
7 Comments:	*This is the first CalTrans FRP project that was publicly advertised and awarded based on low bid.*

Section III Evaluations and Assessments

3-1 Evaluations:	*None after acceptance; 1-1/2 years in place; no special req'ts for bridge inspection.*
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Follow-up to get specs & any other materials or reports needed. Requested 6/27; not received as of 7/7.*

No.: *2* Interviewee: *Jay Thomas* Title: Affiliation: *Structural Preservation Systems* Address: *Springfield, VA*	Phone: *443-271-7100* E-mail: *jthomas@structural.net* Date Surveyed: *7/7* Notes: *References co-authored with Tarek Alkhraji*

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Manager of Strengthering Div -- design/build; upgrades -- new liveload+ Mostly buildings.*
2 Info sources	
3a Materials specs?	
3b Const procedures?	
3c QC procedures?	
3d Perf assess proc?	
4 Application?	
5 Purpose?	
6 Condition?	
7 Specific cond?	
8a Satis service life?	
8b Satis other methods?	
9a Overall satis?	
9b Recommend?	
10 Ideal conditions?	
11 Limitations?	

Section II Implementation Protocols

1 Entities?	
2 If res or univ?	
3 If supplier?	
3 Matls supplier?	
3 Supplier contact?	
4 If engrg firm?	
5 If applic contr?	

6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Very knowledgeable contact; suggests: MO, FL, U of Manitoba, Mark Green at Queen U; Rizkala at NC State*

No.: *3* Interviewee: *Charles Sikorsky* Title: Affiliation: *CalTrans* Address: *Sacramento*	Phone: *916-227-8759* E-mail: *charles_sikorsky@dot.ca.gov* Date Surveyed: *6/7* Notes: *ref by Sheng*

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Several other projects plus two T-girder bridges -- monolithic pours; controlled as a research project. Byron Road -- report available; Watson Wash -- report in July. Sent e-mail.*
2 Info sources	*UCSD; contractors familiar; Mfgrs say it works, but there are a number of questions re. durability; how much increase cap, etc. for which research is needed.*
3a Materials specs?	*Yes & no; have what maintenance uses; will send.*
3b Const procedures?	
3c QC procedures?	*None; QA was a graduate student watching the application.*
3d Perf assess proc?	*No*
4 Application?	*Continuous span, T-beams integrated with deck & columns; monolithic pours; deck was under-designed for current wheel loads; deck beat up; places FRP on surface of deck, between beams; no beam strengthening.*
5 Purpose?	*Strengthening of deck; increase load limits.*
6 Condition?	*Punching failures in deck.*
7 Specific cond?	*Loss of section; concrete failures.*
8a Satis service life?	*Yes*
8b Satis other methods?	*Unclear; shortage of materials because of aircraft construction; advantage due to minimal traffic impact; but, must be carefully placed with QA/QC or it is not worth the cost.*
9a Overall satis?	*Satisfied*
9b Recommend?	*Yes, if done in the right way. Do it right or not at all.*
10 Ideal conditions?	*Note: did a lot of testing of Watson Wash -- in the report. Have a 3 girder structure in lab at SDSU for load testing.*
11 Limitations?	

Section II Implementation Protocols

1 Entities?	*University; in-house design / materials; applications contractor. Done as a research contract.*
2 If res or univ?	*Advisor; Spec matls & details; select supplier; select application contractor; supervise field work; QA/QC; instrumentation; inspection; testing/eval; no studies of service life, cost-eff, long term perf ormance.*

3 If supplier?	
3 Matls supplier?	*Fiber-Toray; Resin - Dow*
3 Supplier contact?	
4 If engrg firm?	*na*
5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	*Instrumented accelerometer grid; looking at global response; no load tests; check out the reports.*
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Requested specs & reports 6/7; reminder 6/27; no response as of 7/7.*

No.: *4* Interviewee: *Mike Mohseni* Title: *Bridge Engineer* Affiliation: *CO DOT* Address: *Denver*	Phone: *303-512-4300* E-mail: *mansour.mohseni@dot.state.co.us* Date Surveyed: *6/8* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*8 - 10 in total; he was involved in 4 to 5; pier caps; column wraps; restoration of an historic arch -- ribs were wrapped with FRP; pier caps completely enveloped.*
2 Info sources	*University, materials supplier; recom by other states; published info; ACI 440 design guidelines; open competition; specifiied FRP but did not specify what fiber to use.*
3a Materials specs?	*Has performance-type spec; will send; requested; received 6/29.*
3b Const procedures?	*Yes*
3c QC procedures?	*Yes*
3d Perf assess proc?	
4 Application?	*Column wraps; under reinforced pier caps; Castlewood Canyon Arch Bridge - wrapped arches.*
5 Purpose?	*Both repair and strengthen; corrosion repair; used carbon fiber plastic rebars - in concrete, wrapped around arch bases.*
6 Condition?	*Severe deterioration; concerned that the arch bridge would be lost.*
7 Specific cond?	*Loss of section; rebar corrosion; salt damage.*
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes; extended arch life by 50 years; spent $300K on FRP.*
9a Overall satis?	*Very satisfied; so far so good; twice inspected since completed.*
9b Recommend?	*Yes, especially for a high bridge.*
10 Ideal conditions?	*Left some gaps at the bottom to allow moisture to escape.*
11 Limitations?	

Section II Implementation Protocols

1 Entities?	*University very involved; matls provider involved - Fyfe; in-house design group; applications contractor -- Restruction; Kiewitt was prime contractor.*
2 If res or univ?	*Advisor worked with DOT research dept; Instrumentation for corrosion; testing after application; tested samples; strength; adhesion; could speak with Ahmad Ardani in Research -- 303-757-9978*
3 If supplier?	
3 Matls supplier?	

3 Supplier contact?	
4 If engrg firm?	
5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	*Post evaluations are 18 month inspections;*
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Very helpful; available for follow-up. Spec + info requested 6/8. Received 6/29.*

No.: *5* Interviewee: *Osman Hag-Elsafi* Title: *Acting Head-Structural Research Group* Affiliation: *NY DOT* Address:	Phone: *518-457-4670* E-mail: *ohag-elsafi@dot.state.ny.us* Date Surveyed: *6/2* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Many column wraps; 5 - 6 cap beams; one complete bridge - prestressed.*
2 Info sources	*Supplier; Contractor (Mitsubishi); published info; willing to try new things and gain knowledge.*
3a Materials specs?	*Yes - guidelines; available thru e-mail request.*
3b Const procedures?	
3c QC procedures?	*Guidelines*
3d Perf assess proc?	*Yes, do load testing; use thermographic methods -- infrared camera; tapping tests to check bond.*
4 Application?	*Reinforced concrete bridge; prestressed concrete T-beams.*
5 Purpose?	*Strengthen and repair; corrosion mitigation; have publications. Cap beam cracking; repair of prestressed beams. Also, Dept will use FRP for cracked aluminum sign structures. Add 15% capacity; follow-up w load tests -- once after, then two more.*
6 Condition?	*Moderate deterioration; a lot of leakage; freeze-thaw cracking.*
7 Specific cond?	*Loss of section; suspected corrosion of rebars; salt damage; freeze-thaw cracking.*
8a Satis service life?	*High cost*
8b Satis other methods?	*Yes; for T-beams costly but worth it to avoid traffic disruption; less costly than replacement.*
9a Overall satis?	*Very satisfied.*
9b Recommend?	*Yes*
10 Ideal conditions?	
11 Limitations?	*Uncontrolled environment; field conditions for mixing materials; need to be able to access; sharp corners a problem; inspection; maintenance; how to repair if damaged.*

Section II Implementation Protocols

1 Entities?	*Materials supplier - Mitsubishi; consulting engineering firm; in-house design / materials; applications contractor. Later, Syracuse Univ. & Cornell Univ. did research on FRP for wearing surface; SUNY-Buffalo studied hybrid concrete; CUNY also did research.*
2 If res or univ?	

3 If supplier?	*Supply only; test data & design values; methods & field work.*
3 Matls supplier?	*Mitsubishi -- supply only; test data +; methods & field work; select application contractor; training & certification of installers; all field work; QA/QC; other was in-house.*
3 Supplier contact?	
4 If engrg firm?	
5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	*4 - 5* *na* *4* *an issue; inadequate* *3* *5* *5* *4* *5* *5*
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	*Pre-repair, materials selection, qualifications in-house;*
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	*Vvisual, sound & infrared imaging; inspection for missed spots, etc; lab tests; pull-out tests; full scale load tests; pull tests on one project; tests on coupon samples.*
3-5 Eval Ongoing:	*Section in guidelines for bridge inspections; 2 years after; will increase to 4 years; load tests before, after & ongoing; behavior of the structure; strain gages.*
3-6a:	*5*
3-6b:	*2, because of state of the technology.*
3-6c:	*5*
3-6d:	*5*
3-6 Other:	
3-7 Other:	*Load testing.*
General Comments:	*Guidelines + reports requested 6/2. Not received as of 7/7.*

No.: *6* Interviewee: *Matt Farrar* Title: *State Bridge Engineer* Affiliation: *ID DOT - Maintenance* Address:	Phone: *208-334-8000 - 4* E-mail: *matt.farrar@idt.idaho.gov* Date Surveyed: *6/7* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*4 total; 2 projects prestressed I-beams*
2 Info sources	*univ indirectly; supplier (SIKA?) to a degree; spoke with applic contractor; sent engrs to seminar in NY; spoke w other DOTs; used ACI guide spec; job was bid out.*
3a Materials specs?	*Have a spec.*
3b Const procedures?	*Have const spec.*
3c QC procedures?	*Design guidelines.*
3d Perf assess proc?	
4 Application?	*Prestressed I -beams.*
5 Purpose?	*Repair; replace existing strength.*
6 Condition?	*Severe deterioration.*
7 Specific cond?	*Loss of section; broken strands.*
8a Satis service life?	*Yes.*
8b Satis other methods?	*Yes. Could do without removing deck. Long plate added to bottom flange + wrapped + mechanical fastener.*
9a Overall satis?	*Satisfied. So far, so good. In place for 3 - 5 years.*
9b Recommend?	*Yes. Seems to be pretty effective, but there has not been another hit.*
10 Ideal conditions?	*Comments: did pull-off tests; pre-loaded bridge to put girder into tension; then patch; trusted design methods; a lot of judgement -- there is a need for clarification in the design codes; used a guide spec from ACI.*
11 Limitations?	

Section II Implementation Protocols

1 Entities?	*Tech supplier; in-house design / materials; applic contr was prime; special provision to pre qualify.*
2 If res or univ?	
3 If supplier?	
3 Matls supplier?	
3 Supplier contact?	

4 If engrg firm?	
5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Requested Spec + on 6/7; Second request 6/27; no response as of 7/7.*

No.: *7* Interviewee: *Jim Gutierrez* Title: *Sr. Bridge Engineer* Affiliation: *CalTrans* Address: *Sacramento*	Phone: *916-227-8256* E-mail: *jim_gutierrez@dot.ca.gov* Date Surveyed: *6/2* Notes: *refer by Sheng*

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*One project for him. Widening of Anderson Road bridge -- error in design calculations -- shear strength deficiency in bent cap.*
2 Info sources	*Univ - yes/no; contractor familiar - yes. Looked at alternatives -- accessibility + ease of construction; presumably also based on prior experience within the Dept.*
3a Materials specs?	*Had column wrap spec; used the same.*
3b Const procedures?	*Supplier provided; contractor experienced.*
3c QC procedures?	
3d Perf assess proc?	*Some pull-off testing.*
4 Application?	*Bent cap.*
5 Purpose?	*Strengthen a new bridge; under-designed (reinforcing).*
6 Condition?	
7 Specific cond?	
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes. Far cheaper than other methods.*
9a Overall satis?	*Agency indifferent.*
9b Recommend?	*Would recommend. Absolutely.*
10 Ideal conditions?	
11 Limitations?	*Would not use if there were no redundancy. Would not use if steel were ruptured.*

Section II Implementation Protocols

1 Entities?	*Matls supplier - Fyfe; in-house design / materials; applications contractor, but same as matls supplier.*
2 If res or univ?	*na*
3 If supplier?	
3 Matls supplier?	*Supply only; physical properties / design values; methods & field work; training of installation personnel; CalTrans Engr responsible for all field work.*

3 Supplier contact?	
4 If engrg firm?	
5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	*4* *na* *4* *3* *na* *4* *4* *2* *3* *Don't know*
6 Comments:	*3 years; no follow-up tests.*
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	*Not for this project.*
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Follow-up for spec on 6/27; no response as of 7/7.*

No.: *8* Interviewee: *Scott Neubauer* Title: *Bridge Rating Engr* Affiliation: *IA DOT* Address: *Ames*	Phone: *515-239-1290* E-mail: *scott.neubauer@dot.iowa.gov* Date Surveyed: *4/29* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes, a dozen -- limited. Minor collision damage to pre-stressed beams.*
2 Info sources	*Iowa State U; supplier somewhat -- SICA, Embrace. Started with university; both fiber wrap & composite.*
3a Materials specs?	*Spec for prestressed only; spec requested & received.*
3b Const procedures?	*No; most work with DOT forces.*
3c QC procedures?	*Yes, there is an in-house manual. Did load tests before and after. Univ involved with early projects.*
3d Perf assess proc?	*No. We haven't done enough to be convinced that it works.*
4 Application?	*Prestressed I-beams.*
5 Purpose?	*Repair of damage.*
6 Condition?	*Severed strands; collision damage.*
7 Specific cond?	*Loss of section; strand exposure / damage. Other projects -- wrapping columns.*
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes, so far. First ones in 1999 - 2000; first one was hit again.*
9a Overall satis?	*Satisfied, but has not been long enough to be sure.*
9b Recommend?	*Yes, depending on the extent of damage.*
10 Ideal conditions?	*See strands.*
11 Limitations?	*Limited if significant portion of core concrete is lost.*

Section II Implementation Protocols

1 Entities?	*Univ; matls supplier; in-house design / materials. Doing with state forces and a local contractor (possibly for support, etc.)*
2 If res or univ?	*Univ was initial designer; spec matls & details; input to tech suppl; initial super; QA/QC; instrum; initial inspect; test & eval. DOT wrote spec provision for subseq jobs; No studies, but plan to. Terry Wipf at ISU 515-294-9501*
3 If supplier?	*Field insp initially; spec & supply; test data & design values; methods & field work; inspect after; testing & eval after*
3 Matls supplier?	

3 Supplier contact?	*1st - Sika Carbodure System; 2nd Mbrace Master Builders.*
4 If engrg firm?	
5 If applic contr?	*Had contractor on first job, but not speciality contractor.*
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	*4* *5* *4* *na* *na* *3* *4* *3* *3*
6 Comments:	
7 Pref. mgmt approach:	*Done a few. Central office lead; no set process; use state forces.*
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	*Pre-repair cond - yes, judgement; Pre-approvals of materials; use state forces to install.*
3-2 Pre Conditions:	*Inject epoxy; grind smoothe; forming & patching; surface prep covered in spec; confident with self-inspection by crews; self inspection of finish coat*
3-3 Applic Proced:	*yes, inspection crews do. Did load testing after 7 days.*
3-4 Eval After:	*univ did this on 3 to 4 bridges -- before & after load tests.*
3-5 Eval Ongoing:	*Yes, with biannial inspection. Delamination -- not found; stain; damage; damaged gel coat; paint coat; biennial; before & after for 3 - 4 bridges; stress test; measure strain in members.*
3-6a:	*4*
3-6b:	*4*
3-6c:	*4*
3-6d:	*3*
3-6 Other:	
3-7 Other:	*Long term - investigate the bond coating.*
General Comments:	*Recvd spec.*

No.: *9* Interviewee: *Joe Lenzini / Tom Domagalski* Title: *Enger of Const for Dist /Br Des Engr Central* Affiliation: *IL DOT* Address: *Carbondale / Springfield*	Phone: *618-549-2171 / 217-782-2125* E-mail: *lenzini@dot.il.gov / domagalski@dot.il.gov* Date Surveyed: *4/28 / 5/31* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*1 project for Lenzini; structure on I-57 in Alexander County*
2 Info sources	*Contacts w matls supplier - Fyfe; contract listed options; contractor made the choice.*
3a Materials specs?	*Yes, have provisions -- in the contract;*
3b Const procedures?	*Combined in spec provisions*
3c QC procedures?	*Combined in spec provisions*
3d Perf assess proc?	*Lenzini doesn't know.*
4 Application?	*Seismic retrofit of round columns - New Madrid fault; 22 span dual structures -- 130 columns*
5 Purpose?	*Seismic upgrade.*
6 Condition?	*na*
7 Specific cond?	*na*
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes*
9a Overall satis?	*Very satis to Satis -- New Madrid fault -- 1826 earthquake*
9b Recommend?	*Yes; yes*
10 Ideal conditions?	*Economy where there is a need to strengthen and not a need to replace.*
11 Limitations?	*Members too deteriorated.*

Section II Implementation Protocols

1 Entities?	*supplier; CE firm; in-house group; applic contractor.*
2 If res or univ?	
3 If supplier?	
3 Matls supplier?	*Spec & supply; test data & values; methods & field work; training / cert;*
3 Supplier contact?	*Fyfe 1999 - 2000; Hexel; Fyfe developed the system.*
4 If engrg firm?	*Prep plans & con docs; seismic anal; not spec*
5 If applic contr?	*Resp for all field work; GC installed - Booker of St. Louis*

6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	*4* *na* *4* *4 good, very low tech.* *na* *na* *4* *na*
6 Comments:	*qualificied applic contractor not an issue; common labor; went well*
7 Pref. mgmt approach:	
7 Comments:	*we are very conservative; this is borderline design-build; new ground; put out alts; a small part of retrofit of structures; ultra conservative.*

Section III Evaluations and Assessments

3-1 Evaluations:	*No eval done; no intent or interest; earthquake retrofit - will wait for the next earthquake.*
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	*Inspected every 2 years; look for surface cracks.*
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Requested spec + from Domagalski; not yet recvd.*

No.: ***10*** Interviewee: *Walid Alaywan* Title: *Sr Structural Engr LTRC* Affiliation: *LA DOTD* Address: *Baton Rouge*	Phone: *225-767-9106* E-mail: *walaywan@dotd.louisiana.gov* Date Surveyed: Notes: *Check web site: www.ltrc.lsu.edu*

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes, 1 research study of load posted T-beams; investigate wet layup & near surface mounting.*
2 Info sources	*university; published lit; consultant from another univ.*
3a Materials specs?	*No, FRP is not on qualified product list (QPL)*
3b Const procedures?	
3c QC procedures?	
3d Perf assess proc?	
4 Application?	*T-beams; Note: opted to stay with high performance concrete on recent study of bulb T-beams.*
5 Purpose?	*Strengther; remove load posting on multi-span (5o' spans?) simple spans.*
6 Condition?	*30 year old bridge, higher ADT, higher loads, now has very good rating.*
7 Specific cond?	
8a Satis service life?	*Unknown -- too early*
8b Satis other methods?	*Unknown -- too early*
9a Overall satis?	
9b Recommend?	
10 Ideal conditions?	
11 Limitations?	

Section II Implementation Protocols

1 Entities?	*univ, more than one supplier; Mrace.*
2 If res or univ?	*Tulane Univ -- Tony Lamanna PI; Nanni as consultant; more than one suppliers; Mbrace.*
3 If supplier?	*Note: does not use supplier data; does data verification in lab; flexure & bond & performance of matrix;*
3 Matls supplier?	
3 Supplier contact?	
4 If engrg firm?	

5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	*Instrument the bridge; add FRP; test; retest & followup every 6 months for 5 years; load rated prior.*
3-2 Pre Conditions:	*Load limited.*
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	*Don: need to follow-up to request info.*

No.: ***11*** Interviewee: *Roger Till* Title: *Engineer of Structural Research* Affiliation: *MI DOT* Address: *Lansing*	Phone: *517-322-5682* E-mail: *tillr@michigan.gov* Date Surveyed: *6/3* Notes: *Tisl*

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes, 3 column wrap; 2 strengthening projects -- 1 box beam and 1 concrete slab*
2 Info sources	*Demonstrations which were follow-ups to university research projects; MSU -- columns; U of M structures*
3a Materials specs?	*Yes; requested*
3b Const procedures?	*Yes*
3c QC procedures?	*Yes*
3d Perf assess proc?	*No, but there is a f year warranty on two cases -- 1 columnl 1 strengthening*
4 Application?	*Prestressed box beams; slab*
5 Purpose?	*Strengthen*
6 Condition?	
7 Specific cond?	
8a Satis service life?	*Satisfactory, but not sure.*
8b Satis other methods?	*Satisfactory; the beam has been in place for four years. For the prestressed box beam there was no other good alternative.*
9a Overall satis?	*Satisfied. Have not gotten thru warranty period.*
9b Recommend?	*Yes*
10 Ideal conditions?	
11 Limitations?	

Section II Implementation Protocols

1 Entities?	
2 If res or univ?	
3 If supplier?	
3 Matls supplier?	
3 Supplier contact?	
4 If engrg firm?	

5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	

No.: *12* Interviewee: *Daniel Hsiao* Title: *Sr Project Mgr for Research* Affiliation: *UT DOT* Address: *Salt Lake*	Phone: *801-965-4638* E-mail: *dhsiao@utah.gov* Date Surveyed: *6/1* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes; 7 sites on one project. Repair columns + bent cap at one site. 8 bridges in one project.*
2 Info sources	*A research project; involved materials supplier and DOT research group. Dept was solicited by suppliers. Also obtained info from published sources and the TRB meeting.*
3a Materials specs?	*Yes, this was years ago; not now used.*
3b Const procedures?	*Yes. Supplier & university -- Pantelides*
3c QC procedures?	*Yes*
3d Perf assess proc?	*No*
4 Application?	*Bent cap / pier cap*
5 Purpose?	*Bith strengthen & repair; corrosion mitigation; seismic upgrade.*
6 Condition?	*Severe deterioration*
7 Specific cond?	*Loss of section; rebar corrosion; salt damage; alkali-silica exposure; severe cracking.*
8a Satis service life?	*Do not know.*
8b Satis other methods?	*Unsatisfactory. Did as a research project; other methods are more cost effective.*
9a Overall satis?	*Cannot answer; too superficial. Varies widely.*
9b Recommend?	*No.*
10 Ideal conditions?	
11 Limitations?	*Too costly; there are more cost effective methods.*

Section II Implementation Protocols

1 Entities?	*University; materials supplier; in-house design / materials group; applications contractor; Caltrans assessment unit; FHWA observed*
2 If res or univ?	*Partly spec of materials & details; partly prepared specs.*
3 If supplier?	
3 Matls supplier?	
3 Supplier contact?	

4 If engrg firm?	
5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	

No.: *13* Interviewee: *Rich Pilcher* Title: Affiliation: *Mo DOT* Address:	Phone: *573-526-4328* E-mail: *Richard.pilcher@modot.mo.gov* Date Surveyed: *7/15* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes, 1 project*
2 Info sources	*Research/university collaaborator*
3a Materials specs?	*No*
3b Const procedures?	*No*
3c QC procedures?	*No*
3d Perf assess proc?	*Yes*
4 Application?	*Conventional slabs*
5 Purpose?	*Repair*
6 Condition?	*Mild deterioration*
7 Specific cond?	*Other-1920s bridge, aged*
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes*
9a Overall satis?	*Satisfied*
9b Recommend?	*Yes*
10 Ideal conditions?	*When the bridge meets the need of the public except needs a high posting and it is cost effective*
11 Limitations?	*MoDOT requires a biennial certified (signed and sealed by PE) report from the local agency to maintain the new posting. When it is all done, you have an old bridge with a few additional years of service. Positive, little to no traffic delays.*

Section II Implementation Protocols

1 Entities?	
2 If res or univ?	
3 If supplier?	
3 Matls supplier?	
3 Supplier contact?	
4 If engrg firm?	

5 If applic contr?	
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	
6 Comments:	
7 Pref. mgmt approach:	
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	

No.: *14* Interviewee: *Nestore Galati* Title: Affiliation: *U of M-R* Address:	Phone: *573-341-6223* E-mail: *galati@umr.edu* Date Surveyed: *6/21* Notes:

Section I Application of Surface Bonded FRP Technology

1 Agency experience?	*Yes, 8 projects*
2 Info sources	*Previous experience; published information; ACI 440 and FIB; center developed design guidelines in AASTHO language*
3a Materials specs?	*Yes*
3b Const procedures?	*Yes*
3c QC procedures?	*Yes*
3d Perf assess proc?	*Yes*
4 Application?	*Pier caps; round piers; rectangular/square pre-stressed beams; conventional RC slabs; arches*
5 Purpose?	*Repair and strengthen*
6 Condition?	*From no apparent to severe deterioration depending on the project*
7 Specific cond?	*Loss of concrete; corrosion of rebar; damage due to salt exposure; damage due to alkali-silica exposure*
8a Satis service life?	*Yes*
8b Satis other methods?	*Yes*
9a Overall satis?	*Very satisfied*
9b Recommend?	*Yes*
10 Ideal conditions?	*Minor concrete deterioration otherwise the repair become expensive.*
11 Limitations?	*DOTs are very concern about the use of FRP technologies even though it is a very consolidate technology.*

Section II Implementation Protocols

1 Entities?	*Research university; suppliers; consulting company; in-house*
2 If res or univ?	*Responsible for all except advisory to the DOT*
3 If supplier?	*Responsible for all*
3 Matls supplier?	*See http://campus.umr.edu/rb2c/people/partners.htm*
3 Supplier contact?	
4 If engrg firm?	*Specification of materials, design of fiber lay-up*

5 If applic contr?	*Responsibility for all field work*
6a: 6b: 6c: 6d: 6e: 6f: 6g: 6h: 6i: 6j:	*4* *5* *5* *4* *4* *5* *5* *4* *5* *4*
6 Comments:	
7 Pref. mgmt approach:	*My experience was at a research/design method. We obtained most successful results when the designer interacted with the contractor in order to avoid misinterpretations of drawings/specifications.*
7 Comments:	

Section III Evaluations and Assessments

3-1 Evaluations:	
3-2 Pre Conditions:	
3-3 Applic Proced:	
3-4 Eval After:	
3-5 Eval Ongoing:	
3-6a:	
3-6b:	
3-6c:	
3-6d:	
3-6 Other:	
3-7 Other:	
General Comments:	

Appendix C: Definitions and Acronyms

The following definitions and acronyms used in this report are primarily taken from ACI 440.2R-02 and NCHRP Report 514 with some changes:

AFRP – Aramid fiber-reinforced polymer.

Aging – The process of exposing materials to an environment for an interval of time.

Alkalinity – The condition of having or containing hydroxyl (OH-) ions; containing alkaline substances. In concrete, the alkaline environment has a pH above 12.

Anchorage – A device at the ends of a FRP bar or laminate that grips the bar or laminate, allowing a minimum of slip and transfers prestressing load from the tendon or laminate to the concrete members.

Bar, FRP – A composite material formed into a long, slender structural shape suitable for the internal reinforcement of concrete and consisting of primarily longitudinal unidirectional fibers bound and shaped by a rigid polymer resin material. The bar may have a cross section of variable shape (commonly circular or rectangular) and may have a deformed or roughened surface to enhance bonding with concrete.

Batch – Quantity of material mixed at one time or in one continuous process.

Bond-critical applications – Applications of FRP systems for strengthening structural members that rely on bond to the concrete substrate; flexural and shear strengthening of beams and slabs are examples of bond-critical applications.

Braiding – A process whereby two or more systems of yarns are interwined in the bias direction to form an integrated structure. Braided material differs from woven and knitted fabrics in the method of yarn introduction into the fabric and the manner by which the yarns are interlaced.

CFRP – Carbon fiber-reinforced polymer (includes graphite fiber-reinforced polymer).

Composite – A combination of two or more constituent materials differing in form or composition on a macroscale.

Concrete substrate – The existing concrete or any cementitious repair materials used to repair or replace the existing concrete. The substrate can consist entirely of existing concrete, entirely of repair materials, or of a combination of existing concrete and repair materials. The substrate includes the surface to which the FRP system is installed.

Contact-critical applications – Applications of FRP systems that rely on continuous intimate contact between the concrete substrate and the FRP system. In general, contact-critical applications consist of FRP systems that completely wrap around the perimeter of the section. For most contact-critical applications the FRP system is bonded to the concrete to facilitate installation but does not rely on that bond to perform as intended. Confinement of columns for seismic retrofit is an example of a contact-critical application.

Creep-rupture – The gradual, time-dependent reduction of tensile strength due to continuous loading that leads to failure of the section.

Cross-link – A chemical bond between polymer molecules.

Cure of FRP systems – The process of causing the irreversible change in the properties of a thermosetting resin by chemical reaction. Cure is typically accomplished by addition of curing (cross-linking) agents or initiations, with or without heat and pressure. Full cure is the point at which a resin reaches the specified properties. Undercure is a condition where specified properties have not been reached.

Curing agent – A catalytic or reactive agent that causes polymerization when added to a resin. Also called hardener or initiator.

Cure time – The time necessary to cure a thermosetting resin system, thermoset-based composite, or prepreg at a given temperature.

Debonding – A separation at the interface between the substrate and the adherent material.

Degradation – A decline in the quality of the mechanical properties of a material.

Delimination – A separation along a plane parallel to the surface, as in the separation of the layers of the FRP laminate from each other.

Development length, FRP – The bonded distance required for transfer of stresses from the concrete to the FRP so as to develop the strength of the FRP system. The development length is a function of the strength of the substrate and the rigidity of the bonded FRP.

Durability, FRP – The ability of a material to resist weathering action, chemical attack, abrasion, and other conditions of service.

E-glass – A family of glass with a calcium alumina borosilicate composition and a maximum alkali content of 2.0%. A general-purpose fiber that is used in reinforced polymers.

Epoxy – A thermosetting polymer that is the reaction product of epoxy resin and an amino hardener.

Epoxy resin – A class of organic chemical-bonding systems used in the preparation of special coatings or adhesives for concrete as binders in epoxy-resin mortars and concretes.

Fabric – Arrangement of fibers held together in two dimensions. A fabric can be woven, nonwoven, knitted, or stitched. Multiple layers of fabric may be stitched together. Fabric architecture is the specific description of fibers, directions, and construction of the fabric.

Fiber – Any fine thread-like natural or synthetic object of mineral or organic origin.

Fiber, aramid – Highly oriented organic fiber derived from polyamide incorporating into an aromatic ring structure.

Fiber, carbon – Fiber produced by heating organic precursor materials containing a substantial amount of carbon, such as rayon, polyacrylonitrile (PAN), or pitch in an inert environment.

Fiber, glass – Fiber drawn from an inorganic product of fusion that has cooled without crystallizing. Types of glass fibers include alkali resistant (AR-glass), general purpose (E-glass), and high strength (S-glass).

Fiberglass – A composite material consisting of glass fibers in resin.

Fiber-reinforced polymer (FRP) – A general term for a composite material that consists of a polymer matrix reinforced with cloth, mat, strands, or any other fiber form. See **Composite**.

Filament – See **Fiber**.

Filler – A relatively inert substrate added to a resin to alter its properties or to lower cost or density. Sometimes the term is used specifically to mean particulate additives. Also called extenders.

Fire retardant – Chemicals that are used to reduce the tendency of a resin to burn; these can be added to the resin or coated on the surface of the FRP.

Flow – The movement of uncured resin under pressure or gravity loads.

FRP – Fiber reinforced polymer; formerly, fiber-reinforced plastic.

GFRP – Glass fiber-reinforced polymer.

Glass fiber – An individual filament made by drawing or spinning molten glass through a fine orifice. A continuous filament is a single glass fiber of great or indefinite length. A staple fiber is a glass fiber of relatively short length, generally less than 17 in. (0.43 m), the length related to the forming or spinning process used.

Glass transition temperature (T_g) – The midpoint of the temperature range over which an amphoras material (such as glass or a high polymer) changes from (or to) a brittle, vitreous state to (or from) a plastic state.

Grid, FRP – A two-dimensional (planar) or three-dimensional (spatial) rigid array of interconnected FRP bars that form a continuous lattice that can be used to reinforce concrete. The lattice can be manufactured with integrally connected bars or made of mechanically connected individual bars.

Hardener – 1) a chemical (including certain fluosilicates or sodium silicate) applied to concrete floors to reduce wear and dusting; or 2) in a two-component adhesive or coating, the chemical component that causes the resin component to cure.

Hybrid – A combination of two or more different fibers, such as carbon and glass or carbon and aramid, into a structure.

Impregnate – In fiber-reinforced polymers, to saturate the fibers with resin.

Interface – The boundary or surface between two different, physically distinguished media. On fibers, the contact area between fibers and coating/sizing.

Interlaminar shear – Shearing force tending to produce a relative displacement between two laminate in a laminate along the plane of their interface.

Laminate – One or more layers of fiber bound together in a cured resin matrix.

Layup – The process of placing the FRP reinforcing material in position for molding.

Mat – A fibrous material for reinforced polymer, consisting of randomly oriented chopped filaments, short fibers (with or without a carrier fabric), or long random filaments loosely held together with a binder.

Matrix – In the case of fiber-reinforced polymers, the materials that serve to bind the fibers together, transfer load to the fibers, and protect them against environmental attack and damage due to handling.

Micro cracking – Cracks formed in composites when stresses locally exceed the strength of the matrix.

Near surface mounted (NSM) – Alternative repair system, where an FRP bar or strip is inserted and anchored into a precut groove.

PAN – Polyacrylonitrile, a precursor fiber used to make carbon fiber.

Pitch – Petroleum or coal tar precursor base used to make carbon fiber.

Ply – A single layer of fabric or mat; multiple plies, when molded together, make up the laminate.

Polyester – One of a large group of synthetic resins, mainly produced by the reaction of dibasic acids with dihydroxy alcohols; commonly prepared for application by mixing with a vinyl-group monomer and free-radical catalysts at ambient temperatures and used as binders for resin mortars and concretes, fiber laminates (mainly glass), adhesives, and the like. Commonly referred to as "unsaturated polyester."

Polymer – A high molecular weight organic compound, natural or synthetic, containing repeating units.

Polyurethane – Reaction product of an isocyanate with any of a wide variety of other compounds containing an active hydrogen group; used to formulate tough, abrasion-resistant coatings.

Pot life – Time interval after preparation during which a liquid or plastic mixture is to be used.

Precursor – The rayon, PAN, or pitch fibers from which carbon fibers are derived.

Prepreg – A fiber or fiber sheet material containing resin that is advanced to a tacky consistency. Multiple plies of prepreg are typically cured with applied heat and pressure; also preimpregnated fiber or sheet.

Pultrusion – A continuous process for manufacturing composites that have a uniform cross-sectional shape. The process consists of pulling a fiber-reinforcing material through a resin impregnation bath then through a shaping die where the resin is subsequently cured.

Resin – Polymeric material that is rigid or semirigid at room temperature, usually with a melting point or glass transition temperature above room temperature.

Roving – A number of yarns, strands, tows, or ends of fibers collected into a parallel bundle with little or no twist.

Sheet, FRP – A dry, flexible ply used in wet layup FRP systems. Unidirectional FRP sheets consist of continuous fibers aligned in one direction and held together in-plane to create a ply of finite width and length. Fabrics are also referred to as sheets.

Shelf life – The length of time packaged materials can be stored under specified conditions and remain usable.

S-N curve – The graphical plot of the repeated load (stress) along a vertical axis versus the number of cycles to fatigue failure on the horizontal axis.

Stress concentration – The magnification of the local stresses in the region of a bend, notch, void, hole, or inclusion, in comparison to the stresses predicted by the ordinary formulas of mechanics without consideration of such irregularities.

Thermoset – Resin that is formed by cross-linking polymer chains.

Thermoplastic – A non-cross-linked polymer capable of being repeatedly softened by an increase of temperature and hardened by a decrease in temperature. Examples are nylon, polypropylene, and polystyrene.

Tow – An untwisted bundle of continuous filaments.

Unidirectional laminate – A reinforced polymer laminate in which substantially all of the fibers are oriented in the same direction.

Ultimate strain – The change in length per unit length corresponding to the tensile capacity.

Vinyl ester – A thermosetting resin containing both vinyl and ester components, and cured by additional polymerization initiated by free-radical generation. Vinyl esters are used as binders for fiber laminates and adhesives.

Weaving – A multidirectional arrangement of fibers. For example, polar weaves have reinforcement yarns in the circumferential, radial, and axial (longitudinal) directions; orthogonal weaves have reinforcement yarns arranged in the orthogonal (Cartesian) geometry, with all yarns intersecting at 90 degrees.

Wet layup – A method of making a laminate product by applying the resin system as liquid when the fabric or mat is put in place.

Wet-out – The process of coating or impregnating roving, yarn, or fabric in which all voids between the strands and filaments are filled with resin; it is also the condition at which this state is achieved.

Witness panel – A small field sample FRP panel, manufactured on-site in a noncritical area at conditions similar to the actual construction. The panel can be later tested to determine mechanical and physical properties to confirm expected properties of the installed FRP laminate.

Yarn – An assemblage of twisted filaments, fibers, or strands, formed into a continuous length that is suitable for use in weaving textile materials.