T. J. Wipf, F. W. Klaiber, E. J. Raker

Effective Structural Concrete Repair
Volume 3 of 3

Evaluation of Repair Materials for Use in Patching Damaged Concrete

March 2004

Sponsored by the
Iowa Department of Transportation
Highway Division and the
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Iowa DOT Project TR - 428

Iowa Department of Transportation

Final

REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil, Construction and Environmental Engineering

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General Abstract

Structural concrete is one of the most commonly used construction materials in the United States. However, due to changes in design specifications, aging, vehicle impact, etc. – there is a need for new procedures for repairing concrete (reinforced or pretressed) superstructures and substructures. Thus, the overall objective of this investigation was to develop innovative cost effective repair methods for various concrete elements. In consultation with the project advisory committee, it was decided to evaluate the following three repair methods:

- Carbon fiber reinforced polymers (CFRPs) for use in repairing damaged prestressed concrete bridges
- Fiber reinforced polymers (FRPs) for preventing chloride penetration of bridge columns
- Various patch materials

The initial results of these evaluations are presented in this three volume final report. Each evaluation is briefly described in the following paragraphs. A more detailed abstract of each evaluation accompanies the volume on that particular investigation.

Repair of Impact Damaged Prestressed Concrete Beams with CFRP (Volume 1)
Four full-sized prestressed concrete (PC) beams were damaged and repaired in the laboratory using CFRP. It was determined that the CFRP repair increased the cracking load and restored a portion of the lost flexural strength. As a result of its successful application in the laboratory, CFRP was used to repair three existing PC bridges. Although these bridges are still being monitored, results to date indicate the effectiveness of the CFRP.

Use of FRP to Prevent Chloride Penetration in Bridge Columns (Volume 2) Although chemical deicing of roadways improves driving conditions in the winter, the chlorides (which are present in the majority of deicing materials) act as a catalyst in the corrosion of reinforcement in reinforced concrete. One way of preventing this corrosion is to install a barrier system on new construction to prevent chloride penetration. Five different fiber reinforced polymer wrap systems are being evaluated in the laboratory and field. In the laboratory one, two, and three layers of the FRP system are being subjected to AASHTO ponding tests. These same FRP wrap systems have been installed at five different sites in the field (i.e. one system at each site). Although in the initial stages of evaluation, to date all five FRP wrap systems have been effective in keeping the chloride level in the concrete below the corrosion threshold.

Evaluation of Repair Materials for Use in Patching Damaged Concrete (Volume 3 - *this volume*) There are numerous reasons that voids occur in structural concrete elements; to prevent additional problems these voids need repaired. This part of the investigation evaluated several repair materials and identified repair material properties that are important for obtaining durable concrete repairs. By testing damaged reinforced concrete beams that had been repaired and wedge cylinder samples, it was determined that the most important properties for durable concrete repair are modulus of elasticity and bond strength. Using properties isolated in this investigation, a procedure was developed to assist in selecting the appropriate repair material for a given situation.

Effective Structural Concrete Repair

General Introduction

Structural concrete is one of the most commonly used construction materials in the United States. Due to changes in the design specification for bridges, increases in legal loads, potential for over-height vehicle impacts, and general bridge deterioration, there is need for new procedures for strengthening and/or rehabilitating existing reinforced and prestressed concrete bridges. In this investigation, strengthening and rehabilitating are considered to be specific means of repairing. The problems previously noted occur in the superstructure as well as in the substructure and are commonplace for state bridge engineers, county engineers and consultants.

In the past, several different materials and procedures have been used for strengthening/rehabilitating structural concrete with varying degrees of success. Some of the procedures used may be effective initially, however, they may not be effective long term especially if the deterioration is due to chloride contamination. Thus, research was needed to develop successful repair methods/materials for strengthening/rehabilitating various structural concrete bridge elements.

Overall Research Objectives

The overall objective of this project was to develop innovative repair methods that employ materials which result in the cost effective repair of structural concrete elements. Carbon Fiber Reinforced Polymers (CFRPs) were found to be the most effective material for long term repair. They have shown promise for use in strengthening and/or rehabilitating various bridge elements. These materials have the advantage of large strength/weight ratios, excellent corrosion and fatigue properties, and are relatively simple to install.

To insure the success of this project, a project advising committee (PAC) consisting of members from the Iowa DOT Office of Bridges and Structures and the Iowa County Engineers Association was formed. The research team met with the PAC on six different occasions. During the initial meetings, the numerous problems engineers have with structural concrete bridge elements were discussed. In later meetings, the research team proposed some potential solutions to the problems previously noted. The outcome of the last PAC meeting was that the following three repair methods should be investigated:

1.) Evaluation of CFRP for use in repairing/strengthening damaged prestressed concrete bridges,
2.) Evaluation of FRP for preventing chloride penetration into bridge columns,
3.) Evaluation of various patch materials.

This project involved a combination of laboratory and field tests. In two cases (1 and 2 noted above), there were laboratory investigations prior to investigating the procedure/material in the field in demonstration projects. The procedures/materials used in the demonstration projects will be periodically inspected until the end of the contract which is Dec., 2008. A log noting the date of the inspection, condition of strengthening system, etc. will be kept for each demonstration project. If a significant change in the strengthening system is observed at one of the demonstrate sites, the structure could be tested if such a test would provide additional information on the repair material/system.

Reports

Since there were three unique repair systems/materials investigated in this project, the results are presented in three separate volumes. Laboratory as well as field test results are presented in this three volume final report. Following this initial report, brief interim reports on the demonstration projects will be submitted approximately every two years. At the conclusion of the project (Dec. 2008), a final summary report will be submitted.

As previously noted, each volume of this final report is written independently. Thus, the reader may read the volume of interest without knowledge of the other two volumes. To further assist the readers in their review of this final report:

- Each volume has a unique abstract, summary, and conclusions, which are pertinent to that part of the investigation. Application guides for installing CFRP on damaged prestressed concrete beams and FRP on columns are presented in Volumes 1 and 2, respectively. A general abstract briefly summarizing the entire project is presented at the beginning of each volume. Thus, the three volume report has four abstracts.
- Each volume has a reference list that is unique to that part of the project. A limited number of references have been cited in more than one volume of the final report.
- The three volumes have different authors – the senior members of the research team plus the graduate research assistant(s) who worked on that part of the investigation.

Volume 3 Abstract

Due to the low tensile strength of concrete, when structural concrete elements deteriorate, are subjected to extreme loadings, or react to corroded reinforcing steel, a portion of the concrete separates from the component and results in a void that needs repaired. Although there have been numerous investigations on patching damaged concrete, the majority of these focus on the high strength and rapid set time of the patch material, neither of which guarantee the durability of a repair.

This study evaluated and identified the repair material properties that are important for durable concrete repairs and recommended a method engineers can use to select repair materials.

To select an appropriate repair material, an engineer must be aware of two factors: the repair material's compatibility with the existing concrete, and the repair material application. Manufacturers use a wide variety of tests to determine the strength of their product; this information can often mislead engineers into using a material that is not appropriate for their situation. Therefore, it is essential to understand the material properties that directly affect repairs and the tests used to determine them.

To isolate the material properties that directly affect durable repairs, 36 reinforced concrete beams were damaged and repaired. The repaired beams were loaded to failure during which time the load/deflection behavior and the patch material's ability to remain bonded to the beam was determined. Wedge cylinder samples were also constructed to evaluate the bond strength and the freeze/thaw resistance of the different repair materials.

The performance of the repair materials in the beam and cylinder tests was compared to data reported by manufacturers. It was determined that the most important properties for durable concrete repairs are modulus of elasticity and bond strength. Materials with high moduli of elasticity performed better than those with lower moduli of elasticity. Materials with high bond strength and low coefficients of thermal expansion performed the best in the cylinder tests. In all cases, materials that had properties similar to those of the concrete being repaired performed well.

A procedure was developed to assist in selecting the appropriate repair material for any situation. The procedure is based on key properties isolated in this investigation, and can be modified for essentially any repair situation.

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1. INTRODUCTION AND REVIEW

1.1 General Background

Many of the bridges that are currently in service in the United States and throughout the world have been in place for quite some time. Over the life of the bridge, the structural concrete deteriorates due to service loads and environmental attacks. In cold weather areas, road salts that are used to de-ice the roadway corrode steel reinforcing and cause decreased capacity. In other cases, bridges are out of date due to the increased traffic loads or increased vehicle sizes. Engineers are faced with the problem of making the bridge comply with existing codes. The options are either to replace the bridge at a large cost or repair the existing one. In some cases it is more economical to build a new structure. In other cases, if only a small portion of the structure is inadequate it would be advantageous to have a method to select a proper repair material. This study reviewed the material properties that are most influential in designing a durable repair and procedures for selecting a material with the most desirable properties.

1.2 Objective of Study

The research in this project was performed to determine the major factors that control the effectiveness of a repair patch placed on an impact damaged concrete bridge girder or damaged footing. The end goal is to provide engineers with a design method for selecting the correct patch material and correct application procedures for various materials and damaged concrete girder combinations.

1.3 Research Approach

In order to recommend specific materials for patch repairs, it was essential to demonstrate how the material properties of the patch affected the performance of the patch.

To isolate the specific material properties that directly affect the effectiveness of a patch, several damaged beams were cast and then repaired with five repair materials with a variety of material properties. The beams were cast with an insert in the bottom of the beam to simulate impact damage. Three additional beams cast without an insert, and used as control beams. Half of the beams, including all of the control beams, were tested to their ultimate capacity to determine the load/deflection behavior of the repaired beams. The other half of the beams were loaded to a fraction of their ultimate load to simulate service conditions. After the simulated service load was applied, the patch was loaded on its side. This load was intended to simulate a second impact and give quantitative information about the remaining bond strength of the patch after a service level load.

To determine how the bond between the patch material and the precast concrete is affected during the course of its service life, cylinders were subjected to 110 freeze/thaw cycles and then loaded axially. The precast concrete was formed in the bottom half of a cylinder, but instead of finishing the top flat, the surface was formed at a 30 degree incline. Then repair material was poured in the remaining portion of the cylinder. The axial failure load and failure locations in the inclined cylinder tests were key components of this test. Once the materials were tested the data were analyzed and a method for selecting the best repair material was proposed.

2. LITERATURE REVIEW

2.1 Introduction

There have been many research articles written about patching damaged concrete in the past several years (1-19). The articles present the virtues of high strength concrete repair materials and their rapid set times. Unfortunately, high compressive strength and rapid set times do not guarantee the durability of a repair.

None of the agencies that control concrete standards have any specific design guidelines for concrete repairs. The agencies report different methods to test in-place concrete for delamination or the pull-off strength of a patch. However, there are no industrial standards for the design of these patches. Because of the lack of standards, and lack of the understanding of why patches fail, many patches perform well initially, but fail after time due to compatibility problems between the patch and the substrate. Several researchers have started to investigate the durability of repair patches. The following summarizes the findings of several of these articles.

2.2 Reviewed Articles

2.2.1 Laboratory and Field Evaluation of Required Material Properties for Concrete Repairs (1)

This study investigated the material properties necessary to ensure a successful concrete repair. The study had a laboratory portion and a field portion. The laboratory portion focused on testing the material properties important to quality, durable concrete repairs. For this project, the repair materials were classified according to the type of primary binder present. There were five categories used: 1) portland cement concrete (PCC); 2) magnesium phosphate concrete (MPC); 3) epoxy polymer concrete (Epoxy PC); 4) methyl methacrylate polymer concrete (MMA PC); and 5) latex-modified concrete (LMC). Because many factors have an effect on property values, Table 2.1 and Table 2.2 only represent approximate values within each material property category. The tables show general trends in material properties between different material categories.

Table 2.1. Typical substrate and repair material properties.

Property	Material Category		
	Plain Cementitious Mortar	Polymer-Modified Cementitious Mortar	Resin Mortar
Compressive Strength (ksi)	2.9-7.3	4.4-8.7	7.3-14.5
Tensile Strength (ksi)	0.3-0.7	0.7-1.4	1.5-2.2
Modulus of Elasticity (10^3 ksi)	3-5	2-4	1-2
Coeff. of Thermal Exp. (10^{-6}/°F)	10	10-20	25-30
Water Absorption (% by mass)	5-15	0.1-0.5	1-2

Compressive and flexural tests were performed when the specimens were 1, 7, and 28 days in age. The cementitious-based materials performed extremely well in comparison with the other materials tested. The two highest 28-day compressive strengths were both neat portland cement concretes (PCC). The polymer-based materials reached their ultimate strengths the quickest. It took only 1 day for the MMA and 7 days for the epoxies to obtain strengths that were extremely close to their 28-day strengths. The polymer-based materials all had a much higher flexural strength than the other materials tested. This is especially true for the 1-day strengths, where their flexural strengths were 3 to 4 times larger than the other materials. For most of the PCC, MPC, and LMC materials tested, their 28-day flexural strengths were between 1,000 and 1,600 psi, while the epoxies and MMA materials had

Table 2.2. Typical cementitious repair material properties.

Property	Material Category					
	PCC	Latex-Modified		Magnesium Phosphate	Epoxy Polymer	MMA Polymer
		Neat	Extended			
Compressive Strength (ksi)	5.0	3.6	5.6	8.7	12.3	12.3
Modulus of Elasticity (10^3 ksi)	3.8	2.5	2.5	3.2	1.6	2.0
Drying Shrinkage(%)	.05-.1	.05-.1	.025	.025	.025	.025

strengths in the range of 2,750 to 3,200 psi. The moduli of elasticity for the polymer-based materials were much lower than those for the other materials tested. The values ranged from 1.45 x 106 to 4.35 x 106 psi depending on the amount of aggregate that was added to the mix. When more aggregate is added, the modulus of elasticity increases. The coefficients of thermal expansion for the polymer-based materials were two to three times larger than those of the other materials tested. The polymer-based materials had values in the range of 12 x 10-6 to 24 x 10-6 in./in./ °F. The PCC, MPC, and LMC materials all had coefficients of thermal expansion comparable to that of typical substrate. In general, their measured values were in the range of 4 x 10-6 to 8 x 10-6 in./in./ F, which is only slightly larger than the value of 6 x 10-6 in./in./°F measured for typical substrate. Most of the materials tested had peak shrinkage strains within the range of 500 to 1,000 microstrains. Only the MMA PC fell out of this range, with its much larger value of 2,000 microstrains. For the other materials, there should not be any shrinkage-related problems if the manufacturer’s curing technique is followed.

A significant portion of this project involved evaluating the bond strength of various types of repair materials. It was found that the majority of the materials tested did not lose significant bond strength due to thermal cycling. These results were obtained by coring samples and conducting pull-off tests after 4 to 6 weeks of thermal cycles. Even though the materials did not lose bond strength due to thermal cycling, the bond strength did vary from material to material. The PCC and MPC material typically had pull-off strengths of 305 psi, while the epoxy PC, MMA, and LMC materials have initial pull-off strengths of about 508 psi. During the pull-off test, failure most commonly occurred at the bond interface. This was explained by the fact that cracks invariably exist at the paste-coarse aggregate interface, even in continuously moist-cured concrete. Pull-off failure also occurred within the substrate. This occurred when the epoxy and MMA materials were used, and can be explained by the high bond strength of these materials. The only material to fail during thermal cycling was the Epoxy PC1 neat mix. This failure occurred after the specimen had achieved initial bond strength of 537 psi.

The material properties that appeared to best predict bond strength are flexural strength and modulus of elasticity. The most important findings based on the repair materials evaluated in this project were that materials with higher bond strengths typically have lower modulus of elasticity values and higher tensile strengths than those materials with lower bond strength. The modulus of elasticity did not predict bond strength as well as flexural strength, but as a general trend the materials with higher bond strengths had lower modulus of elasticity values. It was difficult to establish a relationship for the bond strength as a function of thermal cycles. One of the problems was the difficulty in achieving consistent pull-off strength values due to the high variability in the results. Variability was attributed to: 1) operator error in testing, 2) variability in material strength, 3) type and quality of equipment used to perform the tests, 4) rate the concrete was cored and the pull-off test administered, and 5) eccentricity of the cores.

Relative rankings in Table 2.3 are based on the performance of the materials in laboratory tests, and “high” and “low” rankings are based on the desirable values of the repair material. In order to select a suitable repair material, the rankings in each of the critical material properties are added, and the material with the lowest total is selected.

Table 2.3. Comparison of repair material properties by ranking.

Material Name	Material Type	Compressive Strength (high=1)	Flexural Strength (high=1)	Modulus of Elasticity (high=1)	Coefficient of Thermal Expansion (low=1)	Initial Bond Strength (high=1)
PCC	PCC 1 Neat	2	13	9	4	11
	PCC 1 Extended	6	10	6	9	8
	PCC 2 Extended	12	11	5	3	9
	PCC 3 Neat	1	7	3	5	12
MPC	MPC 1 Neat	9	12	2	1	10
	Mpc 1 Extended	5	9	1	7	7
Epoxy PC	Epoxy 1 Extended	11	4	12	13	3
	Epoxy 2 Extended	3	2	10	10	5
MMA	MMA 1 Neat	7	1	13	12	1
	MMA 1 Extended	4	3	7	11	2
LMC	LMC 1 Neat	10	5	8	8	4
	LMC 1 Extended	8	6	4	6	6
	LMC 2 Neat	13	9	11	2	13

2.2.2 Repair Material Properties Which Influence Long-Term Performance of Concrete Structures (13)

Three generic repair materials labeled A, B, and C were used together with a plain concrete mix. The repair materials are single component, bagged materials that only require the addition of water.

Material A was a blend of portland cement, graded aggregates of maximum size 5mm and additives which impart controlled expansion in both the plastic and hardened state while minimizing water demand. It was characterized as high performance and non-shrinkage, and it can be used to reinstate concrete by partial or total replacement. A water/powder ratio of 0.13 was recommended for use and the typical density of the fresh material is 129 lb/ft^3.

Material B was a mineral based cementitious material with no aggregate size particles or additives. It was relatively porous to allow leaching of salts to continue from contaminated concrete after its repair. A water/powder ratio of 0.16 was recommended and the typical density of the fresh material is 98 lb/ft^3.

Material C was a single component cementitious mortar which incorporates advanced cement chemistry, microsilica, fiber reinforcement and styrene acrylic copolymer technology. The result was a rapid hardening, low density, high strength mortar with enhanced polymer properties. The thixotropic nature of the product made trowel application easy in structural repair of voids, rendering and reprofiling of both vertical and horizontal surfaces. The recommended water/powder ratio is 0.16 and the fresh density of the material is 106 lb/ft^3.

Plain concrete mix used for comparison with the repair materials had constituents of ordinary portland cement, fine aggregate, and coarse aggregates of a maximum size of 0.4 in.

The mix proportions (by weight) were 1:2.24:3.22, with a water:cement ratio of 0.56. The cement content was 21.4 lb/ft^3.

Compressive tests were carried out for each repair material and for the plain concrete. Prisms were made for the compressive tests of each material. The flexural strength of prism specimens was determined under four-point bending at the age of 28 days. Two prism specimens of each mix were tested to determine the static modulus of elasticity at 28 days. Prisms were used for compressive creep tests. Two creep tests were carried out for each material at a sustained stress of 30 and 45% of the 28-day cube strength. In order to calculate the net creep strain, shrinkage was measured on separate specimens and deducted from the total strain measured on specimens in the creep apparatus. To measure drying shrinkage and swelling, ten prisms were used to measure deformation. The first datum strain reading was taken at 24 h after casting, and subsequent changes of length were monitored every 3 days for the first 60 days and once a week thereafter. Four different curing environments of varying temperature and humidity were used.

Material A developed strength rapidly and reached a high compressive strength at 28 days. The elastic modulus and modulus of rupture of Material A are much greater than the respective values for the other materials. Repair Materials B, C, and the plain concrete mix have similar elastic moduli and flexural strengths. The shapes of the compressive creep curves for each material are similar. Materials A and B show comparatively low creep strains. They are roughly 15% less than plain concrete. Material C shows the highest creep strains. The creep of repair materials is more sensitive to the stress/strength ratio than plain concrete is. The repair materials show more drying shrinkage than plain concrete. The shrinkage curves of Materials A and B are similar in shape to plain concrete, but Material C shows very rapid shrinkage for the first 20 days followed by a rate of shrinkage similar to the other materials. Shrinkage of specimens cured in water first for 28 days, then stored in air at 62°F and 55% relative humidity is lower than specimens continuously cured at 62 F, 55% relative humidity after demolding. Additionally, shrinkage of repair materials is much more sensitive to relative humidity of exposure compared to plain concrete. This is especially evident in Material C. The most permeable material was Material B. Materials A, C and the plain concrete had similar permeability coefficients.

The long-term cracking at the repair/substrate interface and the long-term load sharing by the repair patch will be primarily controlled by the shrinkage and creep characteristics of the repair materials. High shrinkage repair materials are more liable to develop shrinkage cracking at the interface with the substrate but this can be reduced if the creep characteristics of patch repair are also high. Polymer additives in repair materials show a small decrease in the water permeability but at the same time they increase the long-term shrinkage and creep deformations. Compressive creep strains are greatest for the generic repair mortar which contains styrene acrylic copolymer, compared to the other materials. Drying shrinkage is greatest for the cementitious repair mortar that contains the styrene acrylic copolymer despite the presence of some fiber additives. Shrinkage of repair mortars with polymer admixtures is much more sensitive to the relative humidity than plain concrete is. Material A, which contains aggregate particles, has less shrinkage and creep deformation than the other repair materials without aggregates. Polymers reduced water permeability.

2.2.3 Factors Affecting Bond between New and Old Concrete (8)

The experimental investigation was used to examine the effects of the following parameters on bond strength: (1) the water-cement ratio of a portland cement mortar (consisting of equal parts by weight of dry sand and portland cement); (2) the thickness of the bond layer; (3) the effect of various curing conditions; (4) the effect of wetting the surface of the hardened concrete before application of the portland cement mortar bonding agent; (5) the effect of delay between mixing a copolymer polyvinyl acetate (PVA) bonding agent and its application to hardened concrete; and (6) the effect of painting on PVA (without addition of aggregate and cement) against using PVA in a mortar. The bond surface was kept dry, unless otherwise noted when the portland cement bonding agent was used; it was wet immediately prior to the application of the PVA bond agent following the recommendation of the manufacturer.

The most obvious fact reflected in the data is the difference in strength between the bonds containing PVA and the bonds containing portland cement mortar. Almost all the portland cement mortar bonds were stronger than the PVA bonds. The thickness of the layer applied affected the bond strength. Three different thicknesses, 1/8 , 3/16, and 1/4 in., were applied in lifts. The results show that the 1/8 and 3/16-in. layers were stronger than the 1/4 in. layer. The 1/4-in. sample failed at the bond line, while the others failed outside the bond area. The authors offer no explanation for this phenomenon. The influence of the water-cement ratio is less clear. The ultimate compressive stress for the 0.32 water-cement ratio bond was on average 1,870 psi lower than for the 0.35 water-cement ratio and 1,480 psi lower than for the 0.40 water-cement ratio. A very low water-cement ratio appears to cause a reduction in bond strength. Prewetting the substrate prior to application of the bond layer may be seen to improve the strength slightly. If a PVA bonding agent is used, the ultimate compressive strength decreases by 10% when the bonding agent dries before the repair material is applied. PVA-modified cement mortars yielded higher bond strengths. The ultimate compressive strength of the mortar was highly dependent on the water-cement ratio. When the water-cement ratio varied from the recommended ratio specified by the manufacturer, the strength decreased drastically. Curing conditions also affected the ultimate strength. Two different specimens were cured under different conditions. One sample was cured for 13 days at 100 percent relative humidity and 14 days at 50 percent humidity. Another sample was cured at 100 percent humidity for 27 days. The specimens cured at 100 percent humidity had strengths greater than 90 percent of those of the other specimens. Thus, the curing difference affected the test strengths only to a small degree.

2.2.4 Evaluation of Test Methods for Measuring the Bond Strength of Portland Cement Based Repair Materials to Concrete (7)

The purpose of the research was to evaluate three bond strength test methods for use in screening and selecting repair materials used in concrete repair. Two methods of gripping uniaxial tension specimens were investigated. Also, a modified ASTM C 882 slant shear bond strength test method was conducted. Three repair materials were investigated: 1) 13 to 14-day-old portland cement concrete (PCC) on 80-day-old base PCC; 2) a 7-day-old latex modified concrete (LMC) with an excessive air content on 94-day-old base PCC; and 3) a 10-day-old LMC with a normal air content on 129-day-old base PCC. The test methods were evaluated by analyzing the failure patterns, the magnitude and relative precision of the failure stresses and the differences in the geometry and loading conditions between the test methods. The two tension test methods used were the friction grips method and the pipe nipple grips method. The friction grips method holds the cylinder with the friction between the specimen and a steel pipe that is split longitudinally and clamped tightly around the specimen. The pipe nipple grips method holds on to the cylinder with epoxy between a steel pipe and the specimen. The modified ASTM 882 test investigates the bond between two slant shear specimens by applying a compressive load. One specimen is base concrete and the other is the repair material being investigated.

With the slant shear test method, failure stress is based on the cross-sectional area (7.1 in.^{2}) and on the elliptical bond plane area (14.1 in.^{2}). The failure bond stress based on elliptical bond plane area was used when comparing the failure stress from the slant shear test method with that from the tension test methods. The failure stress, based on the cross-sectional area, was used when comparing the strength of a slant-shear specimen with the compressive strength of a comparable control cylinder. The failure bond stress was calculated per ASTM C 882 by dividing the failure load (P) with the elliptical bond plane area. The nominal shear bond stress [cosine 30° x P/14.1], which acts parallel to the bond plane, however, is lower than the ASTM stress. The percentage of failure surface area which occurred on the bond plane was deemed “clean” when neither the repair material nor the base concrete adhered to the other. When some material remained bonded together, an estimate was made as to the percentage of the surface that was still bonded together.

With the 14-day-old-PCC-repair material, no common failure patterns were evident in either the friction grips tension test methods, or the slant shear test methods. The different failure patterns were not unexpected with the friction grips test method, since the tensile strength of the control repair material specimens was about the same as that estimated for the base concrete. There appeared to be a common failure pattern in the 14-day-old-repair material in the pipe nipple grips test method. The common failure pattern was not expected, since the tensile strength of the 14-day-old PCC repair material was about the same as that estimated for the base concrete.

With all three test methods, there was a common mode of failure in the excessive air LMC repair materials. Of the total amount of material failure, a larger percentage of the failure occurred in the repair material than occurred at the interface between the repair material and the base material. The failure pattern was as expected for the slant shear tests since the average compressive strength of the excessive air LMC control repair material cylinders was below that of the control base concrete cylinders.

There was a common mode of failure in the base concrete with the normal air LMC repair material and with the two tension test methods, especially for the pipe nipple grips method. With each specimen, the percentage of the failure surface which failed in the base concrete in almost all cases exceeded the sum of the percentage which failed in the repair material and the percentage which failed as a “clean” break. With the normal air LMC repair material, all the slant shear specimens had a “clean” brake value of 75% or greater; which means that 75% of the failure surface was free of repair material.

For each of the three repair materials, the failure stress in the slant shear test based on the elliptical failure plane was substantially greater than that for the two tension tests. This substantial difference in failure stress was attributed primarily to the different test geometry, loading and stresses in the slant shear test as compared to the two tension tests. Values of the ratio of the slant shear average failure stress to that of the compressive strength of the base concrete control cylinders (about 4,900 psi) were 0.81 for the 14-day-old PCC, 0.40 for the excessive air LMC, and 0.86 for the normal air LMC. This ratio represents the strength of the slant shear composite specimen relative to the compressive strength of the base concrete control cylinders. A ratio of this nature could be a useful indication of the expected performance of the repair material in service. The average failure stress for the pipe nipple grips test method exceeded that of the friction grips test method for each of the three repair materials. This was explained by the fact that the pipe nipple test had less eccentricity.

Due to the difference in failure stresses reported by the different type of bond tests, it is important to focus on the test data that will most likely represent the conditions that the patch will be exposed to in its service life. The test methods chosen should have geometry, loading conditions, and stress states that are anticipated for the in-service repair material. It is important to remember that the repair has possible failure mechanisms in the base concrete, repair material, and along the bond line. The slant shear test provided more consistent data than the tension tests. The pipe nipple grips tension test method was considered to be the more promising of the two tension test methods because of its higher average failure stress and better relative precision. If the slant shear test is to be used as the criteria for material selection, the actual test should be performed with caution. Both slant shear and control specimens need to be loaded in compression at the same load rate, and the same cross-sectional area needs to be used to calculate the stresses.

2.2.5 Evaluation and Repair of Impact-Damaged Prestressed Concrete Bridge Girders (2)

This study investigated several different methods of repair on an impact damaged prestressed concrete girder. The damaged girder was a removed from the bridge where it was impacted. The girder had several damaged regions that required a variety of repairs. Although, the researchers spliced strands, performed non-destructive evaluations, and formed and pumped some cementitious materials, this summary of the report will only pertain to overhead repair.

Damaged portions of the girder web that consisted of fractured or delaminated material were repaired with vertical and overhead repair mortars. The repair materials were placed without forms, either by troweling, hand packing, or a combination of both. One and two component materials were used.

Two different two-component latex-modified repair mortars were used: 1) Burke V/O, and 2) Renderoc HB2. Each consisted of a 55-lb. package of dry components, and one gallon of a liquid dispersion of acrylic latex used in place of mixing water. The patched areas were 3 to 4 in. deep by 8 to 18 in. wide. Both of the materials were used to also repair a 6-in. deep portion of the flange. The web repairs were hand-packed and then ground smooth, while the flange repairs were performed with partial formwork in one location and without any forms in the other location. A scrub coat was applied to the surface of the damaged area before the material was placed. Then the mortar was either troweled or hand-packed into the damaged area. The second type of overhead material chosen was a single-component acrylic latex-modified repair mortar. Acrylic Patch was used on a 1/2 to 3-in. deep patch that was 60 in. by 8 in. in plan. The surface was presaturated for 4 hours and a scrub coat was applied before the repair material was applied. The third type of material applied was EMACO S88CA. It is a silica fume, fiber-reinforced, cementitious repair mortar. EMACO S88CA was used to repair a 1/2 to 4-in. deep patch that was approximately 60 in. by 8 in. in plan. A scrub coat was not applied, but the beam was wet for 24 hours before application and wet cured for 7 days.

Initially both two-component latex-modified repair materials looked similar. They were very dry, but as the acrylic was added, the consistency became very sticky. The researchers tried to repair the first section of the web in one lift using the Renderoc HB2. The weight of the patch pulled it away from the substrate. The result was a 2-in. lift. The next day they finished the repair by applying a scrub coat to the first lift and filling the remainder of the damaged area. Renderoc HB2 was used to repair a portion of the flange. Forms were used to perform this portion of the repair. The researchers were only able to get a 2-in. lift, even with the forms. Burke V/O was used in the same situations as Renderoc HB2. The main difference in terms of application between the Renderoc HB2 and Burke V/O was that the Renderoc HB2 pulled away from the surface more easily, leading to thinner lifts. Acrylic Patch was used to repair part of the web. The consistency of this material was much thinner and less cohesive than the two-component mortars. The working time was much less than the two-component materials (10 minutes) and the material would only stick in lifts of 1/2 in. Cold water was used to increase the working time. EMACO S88CA was much darker than the other materials due to the silica fume additive. When applied thicker than 1 1/2 in., the material tended to sag. This led to single lift repairs.

There was no discussion of the performance of the different repair materials to loading or durability. This report only focused on application techniques.

2.3 General Patch Behavior

The first step in determining the proper repair material and repair application method is to determine the conditions that the patch and the existing concrete will experience over the life of the structure. Important considerations are temperature range, load magnitude and duration, chemical environment, and whether the patch is aesthetic or structural. Different patches will perform better in different conditions depending on the material properties of the patch material. There is no one “magical” repair material. Material selection should be a balance between the material properties of the concrete substrate and repair material and the service conditions the member will experience.

2.3.1 Cleaning and Preparing Concrete Before Repair (19)

Once the service conditions have been determined and the repair material has been selected, the existing surface needs to be prepared so that an adequate bond can be achieved. Different manufacturers of repair materials often have a list of “approved” contractors that have been trained in the application of their product. Additionally, the manufacturer will usually provide a recommended application procedure that includes surface preparations. Whenever possible, it is recommended to use the manufacturer’s method to limit the engineer’s liability on the project. In addition to the surface preparation instructions supplied by the manufacturer, there are industry guides and standards that should be followed. The following lists the appropriate standards and guidelines: (19)

- ASTM Standards forCleaning, Surface Preparation, and Testing
- ASTM D 4258 Surface Cleaning Concrete for Coating
- ASTM D 4259 Abrading Concrete
- ASTM D 4263 Indicating Moisture in Concrete by the Plastic Sheet Method
- ASTM D 4285 Indicating Oil of Water in Compressed Air
- ACI Guidelines “ Guide to Durable Concrete” (ACI 201.2R)
- “Causes, Evaluation and Repair of Cracks in Concrete Structures” (ACI 224.1R)
- International Concrete Repair Institute Guidelines
- No. 03730 Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion

The major concern of surface preparation is surface contaminants. Contaminants may be defined as material, either liquid or solid that has the potential to cause adhesion, curing, and/or application-related problems with coatings or patching materials as applied to concrete (1). When dealing with impact damaged concrete beams, unsound concrete and dust are also concerns. All unsound concrete at the surface of the patch must be removed to insure adequate repair material bond. Damaged concrete should be removed with a hammer and chisel (larger pieces) or a sandblaster (smaller pieces). Never use a jackhammer or a scabbler because these large, heavy impact machines can cause microcracks, or bruising, in the concrete. Loose dust or dirt on the surface is most effectively removed with vacuum cleaning or oil-free compressed air.

At this time it is necessary to distinguish between “surface preparation” and “cleaning”. Cleaning refers to the process of removing solvents and dust. Surface preparation involves removing weakened surface layers, removing laitance, and applying any bonding agent recommended by the manufacturer to provide a surface profile adequate to achieve a good adhesive bond. Cleaning should always be performed before surface preparation and immediately before patch material application. The procedure should be– 1) pre-clean, 2) surface preparation, and 3) final clean (19).

It is essential that the patch and existing concrete systems perform as required in the given service environment (1). The factors that control whether the system will perform as required are material properties and quality construction methods. As the engineer, it is our job to select the proper repair materials based on the requirements of the patch system. Due to the nature of the cementitious, pre-packaged repair materials on the market (high early strength, quick set times, and easy to apply), initial application of the material is not a problem. The key to repairing concrete is guaranteeing the durability of the repair system. In order to do that, the engineer must select a repair material with properties compatible with the substrate. The difficulty with determining the material properties that will insure a durable patch is that manufacturers often do not reveal all of the constituents of the prepackaged repair materials. This presents a problem because different constituents have different material properties and successful repair material selection requires a compromise between the desired material properties. The material properties in question are shrinkage, coefficient of thermal expansion, modulus of elasticity, flexural strength, bond strength, and to a much lesser degree, compressive strength.

Many repair materials are sold to repair engineers based solely on the compressive strength and the rapid set times. This can be explained by the fact in most situations compressive strength is the most important concrete property. However, nearly all impact damage to concrete beams occurs on the bottom of the beam, and therefore in the portion of the beam that is in tension. Because of the location of the damage, compressive strength of the patch only affects the durability and overall effectiveness of the system indirectly. The compatibility of the patch material and substrate, however, is of consequence. Compatibility can be defined as the balance of physical, chemical and electrochemical properties and dimensions between the repair phase and the existing substrate phase of a repair system (1). The difficult decision is to decide what is meant by “compatible”. This thesis will not include a discussion of the chemical properties of the repair systems, but it will include an investigation of the thermal properties, bond strength and stiffness of concrete beams repaired with rapid setting cementitious materials.

When there is a significant difference in the coefficient of thermal expansion between the repair patch and the substrate, problems can occur. The problem is that the two materials try to move relative to each other when there is a temperature change. This movement induces internal stresses within each material. Internal stresses can cause the bond to break and cracks in the substrate or patch. When cracking occurs, several problems arise. Cracks in the repair system may allow water to penetrate to the reinforcement. This water, which may contain chlorides from deicing salts, can cause two problems. Water expands when it freezes, and when it is in concrete, may widen existing cracks. These cracks will lead to the deterioration of the patch, and ultimately to its failure. Water, especially water that contains chloride ions, will cause corrosion in the reinforcement. When the reinforcement corrodes, it loses effective section and also expands. The loss of section obviously decreases structural capacity. As the rust builds on the reinforcement and the reinforcement expands, cracks are formed and the bond between the concrete and the rebar is lost.

Patch longevity is also affected by shrinkage. When the fresh patch material is applied, the concrete substrate has achieved dimensional stability. If the patch material shrinks too much as it cures, large internal stresses can occur. This will break the bond between the two materials. For this reason it is desirable to use a patch material that has low shrinkage. Proper curing can minimize shrinkage. Many manufacturers recommend either wet curing with burlap and a moisture barrier around the patch for 2 to 7 days or a curing compound. Some patch materials, however, are incompatible with curing compounds. Incompatibility is also a problem when there is a mismatch in the modulus of elasticity between the repair and substrate. This is especially critical when the repair material is stiffer than the substrate. The stiffer patch will attract a larger portion of the load. A stiffer patch (higher modulus of elasticity) will not deform as much as the substrate and cause redistribution of the load. The redistribution of the load will focus the stress on the interface between the patch and the substrate. The high stress on the interface will eventually lead to a bond failure of the patch. Ideally, the patch should not be as stiff (lower modulus of elasticity) as the substrate, because it will be in the tension portion of the beam. This will allow the patch to elongate with applied load and decrease stress concentration at the interface. However, the modulus of elasticity of the patch cannot be too low. If it is too low, the patch may sag or creep.

From the above discussion it is apparent that when a repair material is selected, care should be taken to match the material properties of the repair and the substrate. It is possible to use a repair material that has different material properties than the substrate as long as the bond strength is not weakened by the induced internal stresses and no durability problems arise from cracking. The key is to ensure that the internal stresses do not exceed the tensile stresses of the substrate, repair material, or the bond strength of the interface.

3. TEST SETUP

3.1 Repair Materials

All of the material properties given in Table 3.1 are directly from manufacturers' literature. This information was used to select the different materials due to the variety of material properties of each product. Through this thesis, repair materials are only identified as Material A through Material E to conceal the identity of the manufacturer of the various repair materials. The main purpose of this research was to develop a selection process for determining which repair material to use in a given situation and to compare various repair materials.

3.1.1 Material A

Material A repair mortar is a one-component, polymer-modified, shrinkage-compensated product, which contains an integral corrosion inhibitor. The product is ideally suited for patching and/or resurfacing distressed concrete. The lightweight nature of the product allows for excellent building without sagging. The working time is 30 minutes.

3.1.2 Material B

Material B is a rheoplastic, shrinkage-compensated cement-based repair mortar. This one-component product is enhanced with silica fume and fibers to provide high strength and superior performance and a corrosion inhibitor. It is specially designed for structural repairs of concrete or masonry and can be applied vertically or over-head by low-pressure spraying or hand troweling. The reported application time is about 45 minutes.

3.1.3 Material C

Material C is a two-component, polymer-modified, Portland cement, fast setting, non-sag mortar. It is a high performance repair mortar for vertical and overhead surfaces, and offers the additional benefit of FerroGard 901, a penetrating corrosion inhibitor. The application time is approximately 15 minutes after the cement (component B) is added to the latex (component A). Application time is dependent on temperature and relative humidity.

3.1.4 Material D

Material D is a one-component, cementitious ready to use repair mortar. The incorporation of low-density aggregates allows high build applications on vertical and overhead surfaces. Application time is approximately 30 minutes.

3.1.5 Material E

Material E is dry hydraulic cement without any chlorides added. If mixed with aggregate it will produce a high quality concrete with 2,000 psi in one hour. It is available in 50 and 88-lb bags. Almost zero shrinkage results from a 6-in. slump. The working time for Material E is about 20 minutes at 70 degrees Fahrenheit.

Table 3.1 Properties of repair material used in this study reported by manufacturers.

Material	Slant Shear Bond [ASTM 882] (psi)	Coefficient of Thermal Expansion (10^{-6} in./in.°F)	Modulus of Elasticity (10^{6} psi)	Splitting Tensile [ASTM 496] (psi)	Flexural (psi)	f_c' (psi)
A	1,500	5.70	2.00	1,500	900 ASTM 348	5,000
B	3,000	6.30	5.00	900	1,300 ASTM 496	11,000
C	2,200	4.20	4.37	900	2,000 ASTM 293	7,000
D	1,000	-	4.10	-	800 ASTM 293	5,000
E	2,680	8.00	4.22	-	-	7,400

- Missing information not provided by manufacturers.
- Note variety of tests used to report material properties.

3.2 Beam Specimens

The flexural test used in this study involved loading simulated impact damaged concrete beams that were repaired with different repair materials. The purpose of this portion of the study was to determine the flexural strength of the repaired beams and the strength of the bond between the repair patch and the concrete beam. These beams were used to determine the stiffness of the beams with different repair materials and to predict cracking loads. Figure 3.1 shows a typical beam used in this test.

Figure 3.1 Damaged beam after repair material is added.

Beams tested were 6 in. x 12 in. x 8 ft in their original undamaged condition. There were 2-#4 (1/2 in. diameter), 40 ksi steel bars running longitudinally and setting on 1 1/2-in. chairs to ensure standard cover. The beams were cast in standard metal forms in six different pours. The concrete (Table 3.2) used was an Iowa Department of Transportation (Iowa DOT) bridge mix C4, which contains 3/4-in. aggregate and 5-7% air entrainment.

During each of the concrete pours, one person performed slump and air tests. Each of the several 6-in. x 12-in. cylinders that were made for each pour were covered with a plastic baggie and cured at room temperature. All the beams in each pour were covered with visqueen and cured at room temperature in the laboratory for three days. After three days, the visqueen and forms were removed and the beams were stored at room temperature. Wedge cylinders and pours for each cylinder were cured similarly. The 28-day compressive strengths listed in Table 3.2 are increased by 10% and then used as the compressive strength of the base concrete throughout this report. The compressive strength of each pour was increased because the flexural tests were performed several weeks after the 28-day tests were performed.

Table 3.2 Compressive strength data of the different concrete pours.

Pour	Date Poured	3 day*	7 day*	28 day*	% Air	Slump (in.)
1	21-May	2,200	3,650	4,800	7.50	4.25
2	28-May	2,900	3,400	4,525	8.00	5.00
3	14-Jun	2,200	3,570	5,850	6.90	5.25
4	29-Jun	2,350	3,410	6,000	7.40	4.00
5	9-Jul	2,500	3,600	6,070	5.20	5.50

* measured in psi

3.2.1 Flexural Test Specimen Construction

To simulate impact damage and to make the damage as uniform as possible, plaster of paris inserts were cast. Each insert, whose center line was aligned with the center line of the beam, was 18-in. long by 1 1/2-in. deep at its deepest cross-section, and feathered at the ends (Figure 3.2). To make each insert uniform, a 6-in. wide concrete beam was used as a model. The model beam was prepared by chipping away an 18-in. long by 1 1/2-in. deep section with 1/4-in. peaks with a chipping hammer.

Once the beam was prepared, two 2 x 4's were used as the forms for the insert. The 2 x 4's were held in place at their ends by two wood clamps. The inside surface of each 2 x 4 and the damaged concrete were coated with form oil so that the insert would not stick to the concrete. To make each batch of plaster of paris uniform, each batche was mixed in two

Figure 3.2 Plaster of paris insert.

plastic 6-in. diameter by 12-in. high cylinders. One cylinder was filled with plaster and the other was filled with 5 in. of water. The contents of each cylinder were then poured into a 5 gallon bucket. The contents were mixed together by hand for approximately 2 1/2 minutes until the slurry had a smooth consistency. The wet plaster of paris mix was then poured into the mold. The sides of the beam were struck several times with a rubber mallet to vibrate the plaster of paris mix. The top surface of the plaster of paris was finished with a hand trowel. The plaster of paris setup in about 30 minutes. When the insert had cured, the 2 x 4 forms were stripped. The insert was pried out of the mold with wood clamps placed on the side of the insert. By lifting on the wood clamps, the insert was freed, after which the mold was cleaned to remove any debris.

Before the insert could be placed in the bottom of the beam forms, the insert was spray painted to avoid drawing moisture from the concrete in contact with the insert while the beams cured. Often the sides of the inserts had to be filed down to fit into the forms.

A total of thirty-six beams were cast in the manner described above. Three of the thirty-six beams were cast without inserts and were used as control beams to determine the behavior of the beams without damage. Once the beams had cured for 28 days, they were readied for repair. The patch was removed, and the surface of the damaged beams where the patch had been, was scrubbed with a stiff metal brush to remove paint and any plaster of the paris that adhered to the beam.

Since form oil had been applied to the plaster insert, the oil had to be removed from the damaged surface. The oil was removed with a trisodium phosphate (TSP) mixture – 1/4 cup TSP and 1 gallon of warm water. The surface of the damaged beams was scrubbed with the TSP solution until the surface absorbed water (19). Once the form oil was removed, the beam was rinsed with water to remove any TSP remaining on the surface; any remaining debris was removed with oil-free compressed air.

3.2.2 Material Application

A total of six beams were repaired with material A. The damaged surface was soaked over night to a surface saturated dry (SSD) condition. SSD is a common condition referenced by concrete repair material manufacturers. Three beams were positioned with the damaged portion on the bottom to simulate overhead repair conditions. Figure 3.3 shows a worker wetting a beam in its overhead position.

Material A is provided in 55-lb bags and requires 1 gallon of water per bag. One half of a bag was weighed and poured into a plastic 5-gallong bucket. One half gallon of water was poured into a clean 5-gallon bucket. Material A was added at a steady rate to the water while a paddle mixer, attached to a power drill was used to mix the two materials for approximately three minutes. The consistency of the mixing went from a damp powder to a thick paste as the latex admixture was activated. It was tempting to add additional water in the first two minutes of mixing, but it was not recommended. Small amounts of water, as little as a couple of drops, can significantly change the consistency of the repair material once mixing has begun. The damaged surface of the beam was splashed with water again before the repair material was applied.

Figure 3.3 Wetting the damaged surface of a beam specimen.

Material A does not require an epoxy-bonding agent. Instead, a scrub coat is applied with a stiff bristled wire brush. The scrub coat must be worked into the pores of the concrete surface to ensure a good bond between the repair material and the existing concrete. A putty knife was used to apply Material A 1/2 to 3/4-in. lifts (Figure 3.4). If deeper lifts were applied, the material would sag in the middle and peel off the beam. Occasionally, when the patch was approaching its full depth, large chunks of the repair material would fall out. It was often easier to apply the material by hand, but this caused skin irritation and dryness. The key to using Material A was patience. The manufacturer's published set time is 20 minutes, but applying lifts less than an hour apart is not recommended. Material A is a thixotropic material, which means that if agitated, it will return to a plastic state. When the material returns to a plastic state, the working time of the material is increased, and the material has a tendency to peel off the beam. The manufacturer recommends scoring the bottom of each lift to promote bond to the next lift. This was difficult to do because of the thixotropic nature of the material. The surfaces were lightly brushed between lifts to remove some material that had not fully hardened; it took four lifts to repair each beam. It was possible to repair three beams and cast the 12-3-in. diameter cylinders with two bags of material, but there was a significant amount of waste in each batch mixed because of the short pot life and the amount of material that fell onto the ground. It took four lifts to repair each beam.

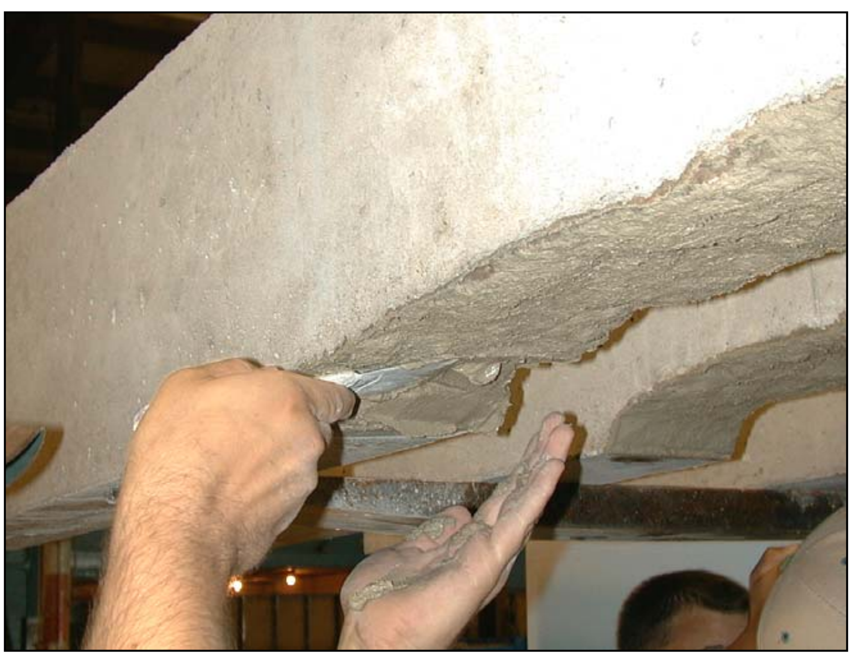


Figure 3.4 Application of repair Material A with a putty knife.

Material B was the second product used to repair the beams. It also comes in 55-lb bags, but does not have the same amount of yield per bag as Material A. It is darker when mixed with water and had visible fibers in the paste. Like many of the repair materials, it was difficult to determine if the material had the correct water/cement ratio. This is important because the bond between the concrete and repair material was very sensitive to the water/cement ratio. If the material was too dry, it setup very quickly and the batch was lost or the repair material would not stick to the beam. A convenient method to determine if the material was ready to be applied was to stick a putty knife into the slurry and then turn it upside down. If the material stuck to the putty knife, it was properly mixed. If the material did not stick, a change was made. If the material was too dry, water was added and the slurry was mixed longer. Sometimes it was determined that there was adequate water, but that the latex had not become active. This determination was made based on the smell of the material. After working with each material, it was possible to identify the smell of the latex. On some of the materials, not Material B, it was possible to add additional water, then wait for the material to gain the consistency necessary for good bond. This increases the slump and shrinkage of the repair material, but sometimes it was the only way to make the material achieve a proper consistency. The high slump material worked well as a scrub coat (Figure 3.5).

The same basic set-up was used for the Material B as was used for Material A. One half of a bag was mixed at a time because there was concern that the power drill would not be able to mix an entire bag. Material B has a longer pot life than Material A, so set time was not a concern. One thousand eighteen hundred fifty milliliters of water were poured into a 5-gallon bucket. The cement was added and mixed for approximately 3 minutes. Material

B was much darker and thicker than Material A and had a thick oatmeal consistency, whereas Material A was grittier. As shown in Figure 3.5 the surface of each damaged beam was dampened and a scrub coat was applied. Without a scrub coat many of the repair materials did not adhere to damaged section of the beam.

Figure 3.5 Worker applying scrub coat with a brush.

It was possible to apply Material B in 1-in. lifts, as it adhered to the concrete surface very well. Figure 3.6 shows workers applying a lift of repair material. It was possible to work with Material B for more than 40 minutes. Because of the longer pot life, better bond, and higher lifts, there was much less material wasted with Material B than with Material A. One bag repaired 3 beams and all of the cylinders.

Material C, a two-component repair material, was the third product used in this study. Instead of adding the aggregate to a given amount of water, the aggregate is added to latex.

The manufacturer provides this latex in one-gallon containers. The normal mix ratio was one gallon of latex per bag of material. Even though Material C was a two part material, the mixing process was nearly identical to the mixing process of the one-component materials. Slightly less than half gallon of latex was measured out and poured into a 5-gallong bucket.

Figure 3.6 Application of repair Material B.

It was determined that less latex than recommended by the manufacturer was required because of the environmental conditions in the laboratory. Half a bag of Material C was added to the latex in the 5-gallon bucket. This mixture was stirred with a power drill and a paddle mixer. This material adhered very well to the base concrete, but each lift did not stick to the previous lift as well as the lifts of other materials stuck to each other. Therefore, it was not possible to build Material C in lifts as high as some of the other materials. Material C was applied in three different lifts due to the low build capacity. See Figure 3.7 for a finished patch.

Material D was the fourth material used and was provided from the same manufacturer as Material C. Latex could be used instead of water, if the application required high bond strength. At the time of repair, it was more important to make the repair as easy as possible, instead of increasing the bond strength, so latex was not used. Material D is more of a specialty product than Material C and is more difficult to acquire from suppliers. This is interesting because, solely on the basis of application, this was the best material used in this project in terms of application. It was possible to apply Material D in up to 1 1/2 in. lifts. The only problem was that it was extremely sensitive to the water/cement ratio used during mixing. All of these materials were very sensitive to the mixing process and amount of water

Figure 3.7 Finished patch on a repaired beam specimen.

added, but Material D was the most sensitive. Both Material C and Material D were extremely abrasive on the skin. The abrasive agent was probably the latex in both materials. Workers had to use surgical latex gloves to protect their hands from serious cracking and chafing.

Material E, the last product used, was more of a cementitious material than the other materials used. It was more of a light tan or crème color, whereas the other materials were more of a concrete gray. Material E was not specifically designed for over-head patching repairs, so the recommended mix ratio on the bag which recommended adding one gallon for each 55-lb bag was adjusted. This mix ratio was used to try to reach a 5 or 6 in. slump. From experience with the other repair materials, it was determined that a 5 or 6 in. slump would be too wet to hand overhead. Initially, 1/3 gallon of tap water was added to half a bag which resulted in the same consistency as the other materials used. However, this material dried out as mixing continued, whereas the other materials became more viscous. More water was added, but the hydration reaction was causing so much head at this point, that the entire batch setup in the bucket is less than 8 minutes. When the next batch was mixed, 3/4 gallon of ice water was added to 1/2 bag of material. This created a much higher slump mix, which had a perfect consistency for a scrub coat. The material was too “runny” to apply overhead for nearly 5 minutes. After 5 minutes, the material started to setup enough to be applied to the beam. It had a total work time of about 15 minutes, with the first 5 minutes of waiting included.

Material E had excellent lift capacity and this allowed the entire beam to be repaired in one lift; it would even stick to you hands if dry. Near the last minute or two of the work time, Material E became graining and difficult to work. Fortunately, once the material lost the ability to be added to the damaged beam, the material was still plastic enough to be finished smooth. When dried, Material E was the closes in color to the base concrete as any of the other materials. It was possible to repair 3 beams with 1 1/2 bags.

3.2.3 Analysis of Flexural Test Specimens

In order to predict the behavior of each repair combination, two different methods were used. The first method transformed all three materials (base concrete, reinforcing steel, and repair material) into an equivalent material. Because the dominant material in terms of are for any beam was the base concrete, all materials were transformed into the base concrete. J The transformation was based on equivalent modulus of elasticity. Once the three different materials were transformed, the moment of inertia of the transformed moment of inertial was calculated. Detailed accounts of the calculations are shown in Appendix A, and the results of the calculations of both methods are presented in Chapter 4.

Theoretical deflection calculations using Method 1 assume the repair material spans the entire length of the beam. However, the patch material only spans 18 in. of the 96-in. total length of the beam. This obviously, is not a very accurate assumption, and thus leads to the second method of analysis. Method 2 models the beam as two materials (base concrete and reinforcing steel) instead of the three used in Method 1. This will neglect the repair material in the deflection calculation, but as will be shown later, this is an acceptable method for modeling the stiffness of the beam because of the small amount of the repair material in the beam compared to the base concrete. The only variable in Method 2 from pour to pour is the compressive strength of the concrete.

Both methods have their flaws. Method 1 over estimates the stiffness of the beams when the repair material has a higher modulus of elasticity than the base concrete and underestimates the stiffness when the repair material has a lower modulus of elasticity than the concrete. Due to the small percentage of the beam's length that has repair material instead of concrete, it would seem that Method 2 would be a better estimate of beam stiffness.

In addition to predicting the stiffness of beams repaired with different materials, it would be informative to have a method to predict the load that causes the beam to crack. Again there were two methods used to determine the cracking load. Method 1 uses the three-material model and Method 2 uses the two-material model. The calculations shown in Appendix A are used to determine the load that causes the beam to crack. At the point on the load/deflection plot where the beam cracks, the slop changes because the beam decreases in stiffness due to the decrease in the moment of inertia. The modulus of rupture of the concrete is used in the cracking moment calculation because it is assumed that the concrete has a lower modulus. The crack was assumed to be a flexural crack in the repair material and not a crack in the bond between the concrete beam and the repair material because the bond strength of the repair material is much stronger than the modulus of rupture of the repair material. This assumption was verified in the flexural tests. Vertical flexural cracks appeared before there was any sign of the patch debonding.

3.2.4 Push Out Shear Test

The beams described for the flexural tests were also tested on their sides to test the pure shear capacity of the patch material. In order to isolate the shear strength of the patch material, a shear load was applied to the side of the patch. Figure 3.8 shows the test setup for the push out shear test. The beam was supported at the ends and centerline; Figure 3.9 shows how the channel supports the beam at the centerline. The support in the vicinity of the loading point coupled with the force applied to the patch subjects the bond between the concrete beam and the repair material to shear. A 20 kip hydraulic cylinder was used to apply load to a 12-in. long W6 steel beam whose bottom flange was cut to a 1-in. width which in turn applied the load to the patch. Care was taken to make sure that the load applied to the patch was purely vertical to insure that the resulting stresses were purely shear stresses.

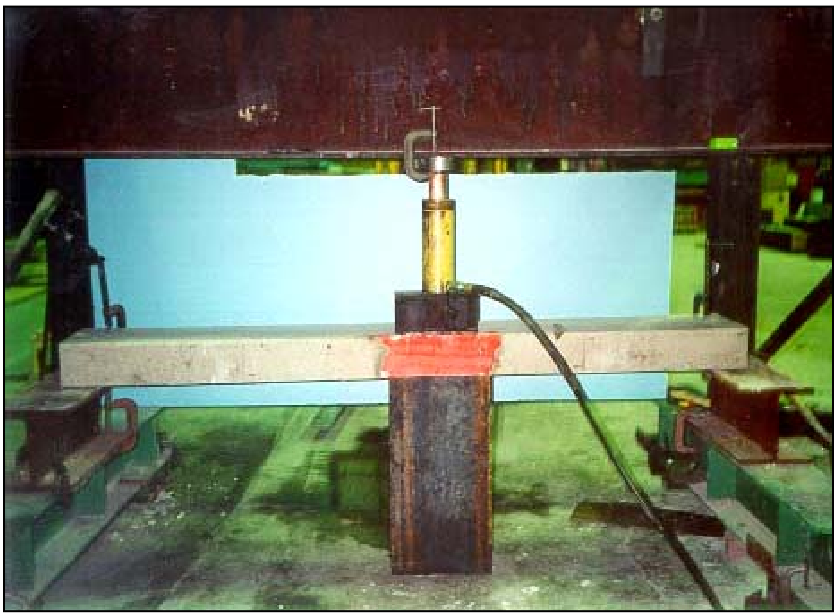


Figure 3.8 Push out shear test setup for one beam specimen.

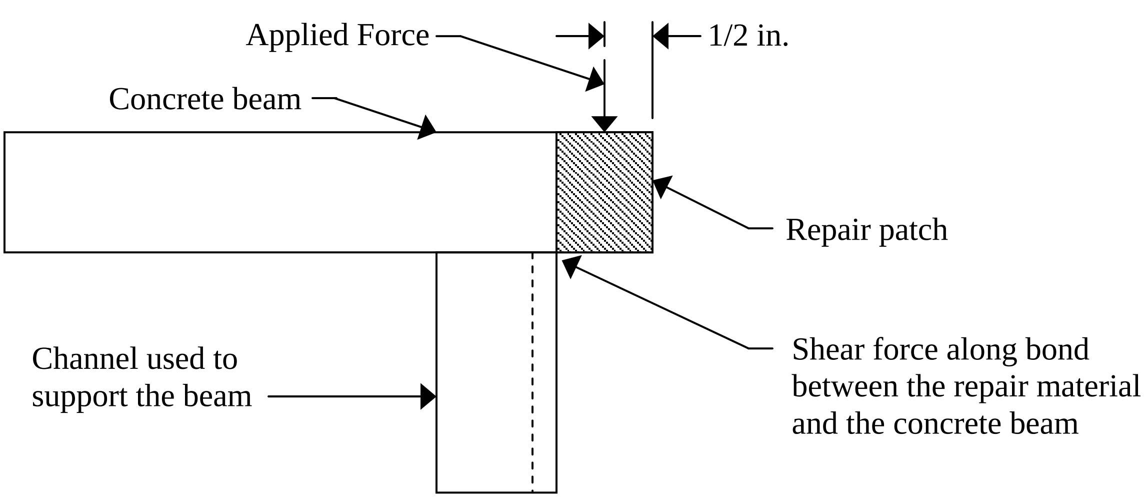


Figure 3.9 Push out shear test specimen supported at the beam centerline.

All the patches in the push out shear test were exposed to flexural stresses by applying a 6,000-pound load at the midpoint of the beam (span length = 96 in.) to simulate service level loads before they were loaded in the push out shear test.

3.2.5 Analysis of Push Out Shear Test Specimens

As discussed earlier, the damaged area of the concrete is 18 in. long by 1 1/2 in. deep by 6 in. wide. Each patch was made from the same mold so that the surface roughness and overall dimensions of each patch was essentially the same. The patch has been modeled (Figure 3.10) as an arc of a circle in profile to calculate the surface bond area.

$$r = \frac{4(1.5)^2 + 1}{8(1.5)} = 27.75\text{in.}$$

$$c = 0.017453(r)37.85 = 18.322 \text{ in.}$$

$$A_s = 18.32{*}b = 109.93 \text{ in}^2$$

where

r = radius　　　　　c = arc length

A_s = shear area　　　b = beam width = 6 in.

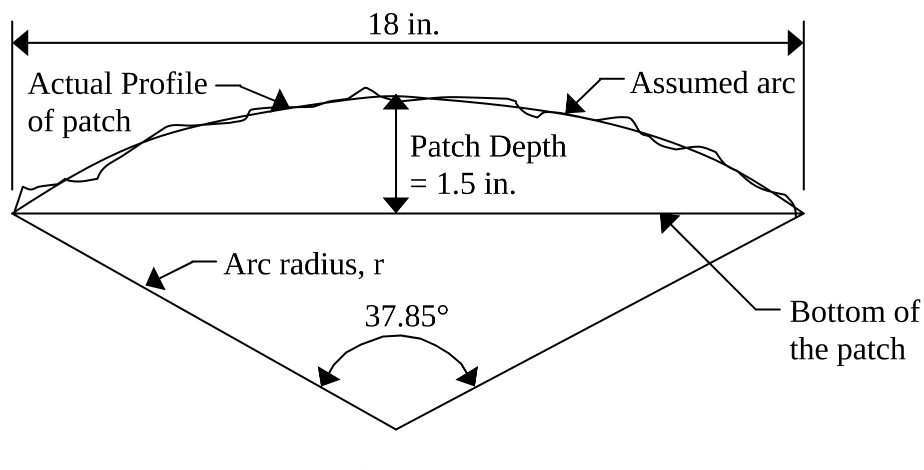


Figure 3.10 Assumed geometry of the patch used to calculate the shear area in the push out shear test.

Once the arc length has been calculated, the shear area can be evaluated. The shear area, A_s, is the arc length multiplied by the beam width. Using this shear area, neglecting the roughened surface, it is now possible to calculate the bond strength of each material in the beam shear test.

3.3 Wedge Cylinder Specimens

In addition to simulating damage in beams, this study tested the shear strength of cylinder samples to determine the bond strength of the repair materials after freeze/thaw cycles. Each combination of concrete and repair material was subjected to two different environments. Once the cylinder cured, six specimens from each repair combination were placed in a freeze/thaw machine. Three additional cylinders from each repair combination were tested on the day that the freeze/thaw cycles started. These provided the strengths of the different repair combinations at zero cycles. Three of the cylinders were removed after 110 cycles and tested in compression. The remaining three cylinders from each repair combination were removed and tested after 122 cycles. Additionally, three cylinders from each combination were prepared without a wedge of base concrete. These three cylinders were used to determine the compressive strength of the repair material alone on day one. The actual curing time for each repair material differed because samples of each material were made at different times.

3.3.1 Cylinder Construction

The cylinders were tested in compression using the slant shear test (ASTM C882-91). The test specifies that the cylinders used should have a diagonally cast bonding area at a 30-degree angle from vertical. Four racks (see Figure 3.11) were built to keep the cylinders at a 30-degree angle from vertical. Each rack was built with a7 in. wide, 8 ft long piece of 1/2 in.

plywood, an 8 ft long 2 x 4, and a 2 ft long 2 x 6. The 2 x 4s were nailed to the plywood with the 3 1/2-in. sides perpendicular to the face of the plywood. One corner of the 2 x 6 was beveled at a 30-degreee angle from the vertical so that the plywood could be nailed to it. Each of the concrete cylinders was rested with the circular face on the 2 x 4 so that the cylinder made an angle of 30-degrees from the vertical. The cylinders were 3-in. in diameter and 6-in. tall. The wedges were made in the fifty concrete pour, and the original empty cylinders were cut with a band saw. All 90 of the cylinders were placed on the four racks. Prior to casting the cylinders, air and slump tests were performed on the concrete. After the tests showed that there was sufficient air entrainment (5.2%), concrete was placed in the wedge shaped cylinders, and extra concrete was screeded off with a trowel. Striking the corner of the cylinder on the ground while keeping the cut surface horizontal consolidated the concrete in the cylinder. Once the concrete had been consolidated, the surfaces were scored

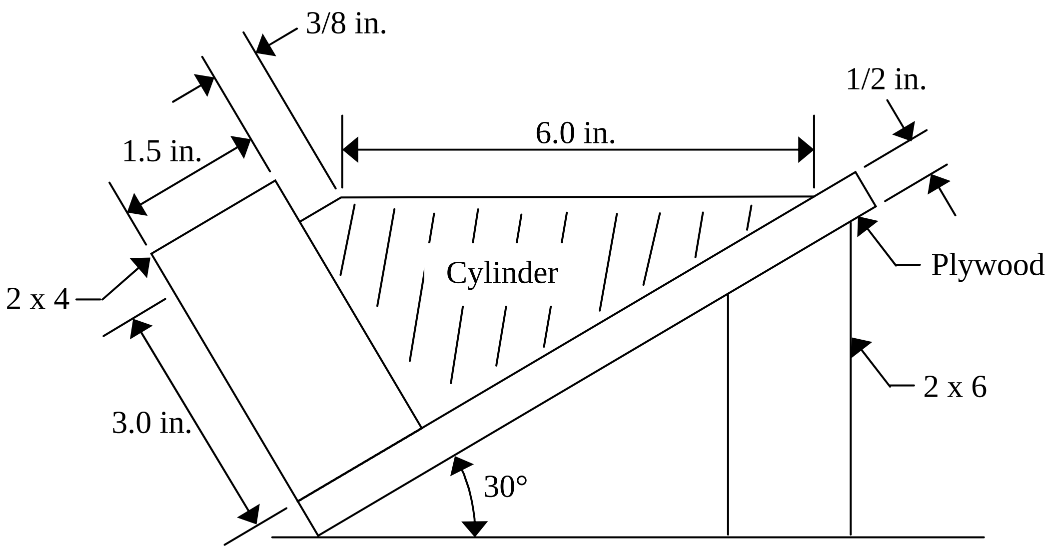


Figure 3.11 Cylinder stand used to make the wedge cylinder samples.

with a trowel to simulate the roughened surface of a damaged beam. A waffle type pattern (cylinder on the right in Figure 3.12) was scored into the future bond surface with the edge of a concrete trowel. Workers tried to keep the grooves as uniform as possible from wedge to wedge. The hardest part of the etching process was etching the feathered end of the wedge. When large pieces of aggregate were in the feathered end, they were removed and the entire process was repeated. The lack of aggregate near the feathered end of the wedge may cause the feathered end of the wedge to be weak. To eliminate this variable in the local strength of the wedge samples, the repaired cylinders were placed in the test machine with the concrete wedge at the bottom and the repair material wedge at the top. The cylinders were covered with visqueen and cured in the laboratory for one week. The first repair cylinders were made 25 days after casting the wedges.

Figure 3.12 Wedge cylinder sample bonding surface.

3.3.2 Addition of Repair Material to Wedges

When it was time to add repair material to the concrete wedges, the wedges were removed from the wedge shaped plastic forms by slicing the plastic form away from the cylinder. After the concrete wedge was removed, it was placed in a new full cylinder. While each specific repair material was being used to repair the beams, one worker placed the repair material in the cylinder. The cylinders and the beams were repaired with the same batch of repair material so that the repair material used in the beams and the cylinders would have the same material properties. The only time that a repair material was prepared differently for the beams and the cylinders was when Material E was used. Material E hardened so quickly, even with the addition of cold water, that it was thought that the material would setup before it could be placed. Therefore, a batch of Material E was mixed specifically to repair the beams and a separate batch was mixed to repair the wedge cylinders.

3*.3.3 Cylinder Test Analysis*

Three full cylinders and three wedge cylinders of each repair material were tested in axial compression at day zero, to test the initial compressive strength. Six cylinders of each repair combination were then subjected to 110-freeze/thaw cycles. The freeze/thaw test was originally intended to go for 200-freeze/thaw cycles. Upon visual inspection of the cylinders at 110 cycles it was observed that the cylinders were starting to degrade. To ensure that the cylinders did not disintegrate in the test machine, a decision was made to remove the cylinders after 110 cycles.

A 2 in. x 6 in. cylinder has a cross-sectional area of 7.1 in^2. A repaired wedge cylinder (see Figure 3.13) has the same cross-section as the full cylinder and the surface area of the interface between the repair material and base concrete is 14.1 in^2. ASTM 666 C governed the proportions of the wedge cylinders. Sample wedges were poured with a smooth surface and it was determined that this was not representative of the surface conditions that would occur on a damaged concrete bridge girder. Therefore, the surface of the interface between the repair material and the base concrete was roughened with a trowel when the base concrete wedges were formed. The roughened surface was a waffle pattern with 1/8-in. to 1/4-in. grooves to provide adequate texture for the bond. By adding grooves to the wedge cylinder, the bond strength measured with this test setup will be different from the reported values by the repair material manufacturers.

Figure 3.13 Repaired wedge cylinder before testing.

Full cylinders were made entirely of repair material and were used to determine the experimental compressive strength of the repair materials. The strength of the full cylinders was evaluated in terms of axial load (lbs.) in terms of ultimate stress (psi). The ultimate stress of full cylinder samples was determined by dividing the ultimate load by the cross-sectional area of the cylinder (7.1 in^2).

Shear cylinders were made of a combination of repair material and concrete. The concrete used for every shear cylinder was from the pour five (f_c' = 6,070 psi at 60 days, 5.5 in. slump and 5.2% air entrainment). The failure stresses were calculated by dividing the ultimate load by the appropriate cylinder area. When the failure was purely compressive, the area used was the cross-section perpendicular to the applied load (7.1 in^2). When the failure was a shear type failure, the failure occurred along the line between the concrete and the repair material. The area used to determine the bond shear stress along the bond line is 14.2 in^2.

4. TEST RESULTS

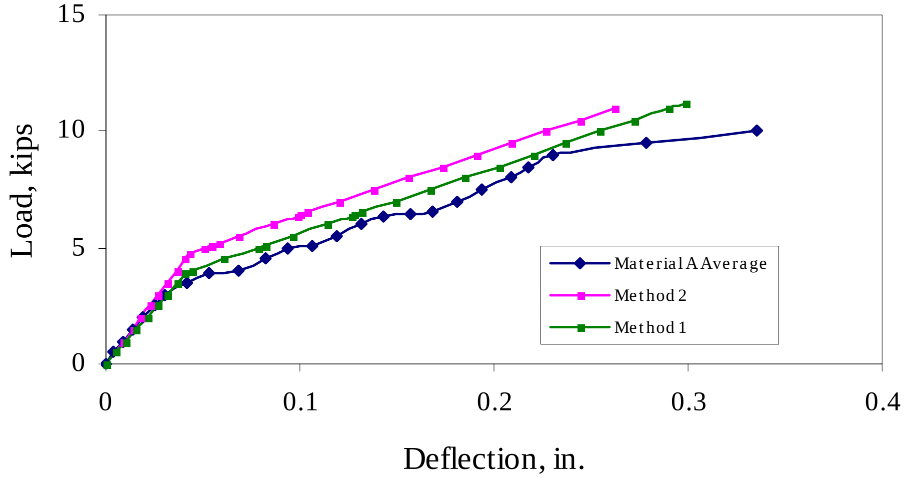
4.1 Beam Specimen Flexural Test

The flexural test portion of this study was performed to evaluate the performance of repair materials used to repair damaged concrete beams. The repair materials that were selected had various compressive strengths, modulus of elasticities, and bond strengths.

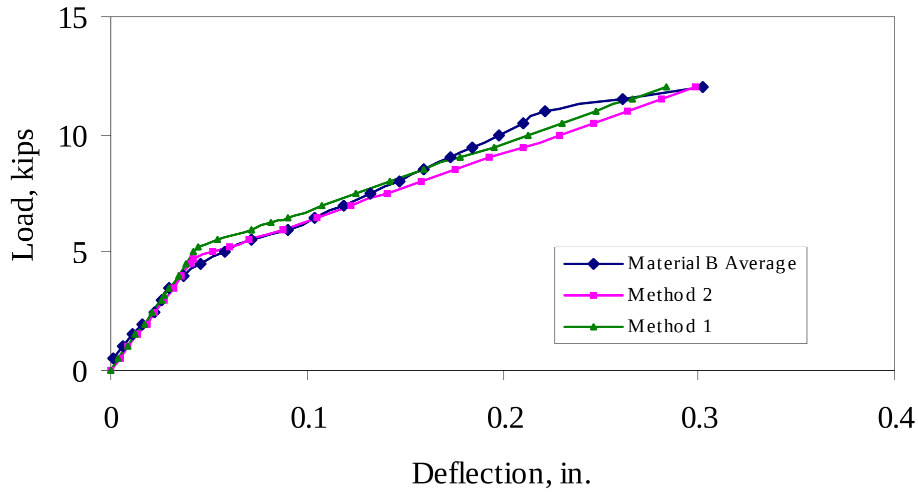
4.1.1 Load/Deflection Plot Information

Typical load/deflection plots for Materials A-E are shown in Figures 4.1a-f, respectively. Each individual plot has three lines. One line shows the average experimental load/deflection behavior of the three repaired beams tested for each repair material. The other two lines are the predicted load/deflection results based on Method 1 (beams with three materials) and Method 2 (beam with two materials), respectively, which were discussed in Chapter 3. Figure 4.1f shows the load/deflection behavior for an unrepaired beam, which will be referred to as the control beam. Table 4.1 presents a comparison of the moments of inertia calculated using Methods 1 and 2. This table shows that the moment of inertia of each beam/repair system does not vary much between the two methods (i.e. M2/M1 ratio close to one for all materials). This is significant because it means that the repaired beams can be analyzed with standard reinforced concrete elastic analysis techniques.

The load/deflection plots for every repair material and control beams show classic reinforced concrete (R/C) behavior. There are two regions of behavior; the first region shows the uncracked behavior of the beams. In the first region, the beams have not cracked, and the deflection is inversely proportional to the uncracked moment of inertia. At the cracking load, Pcr, and corresponding cracking moment, Mcr, which causes the extreme

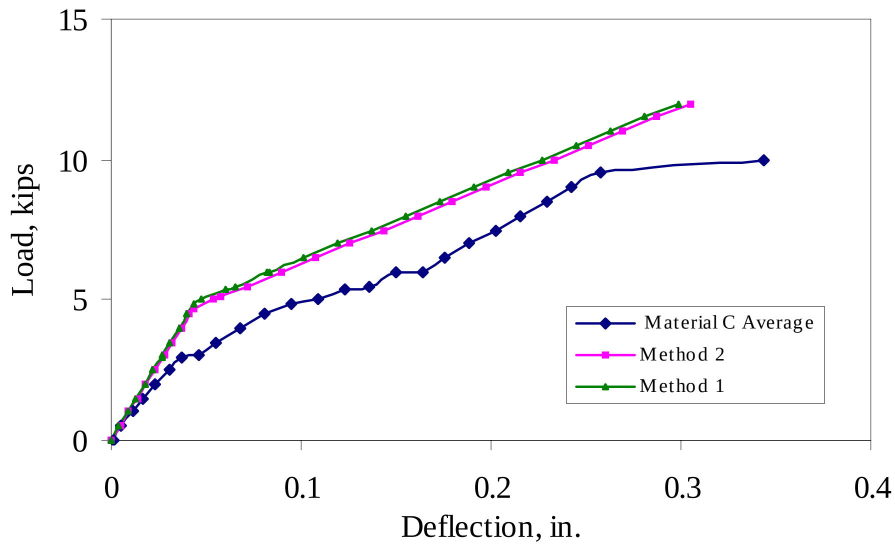


a.) Material A

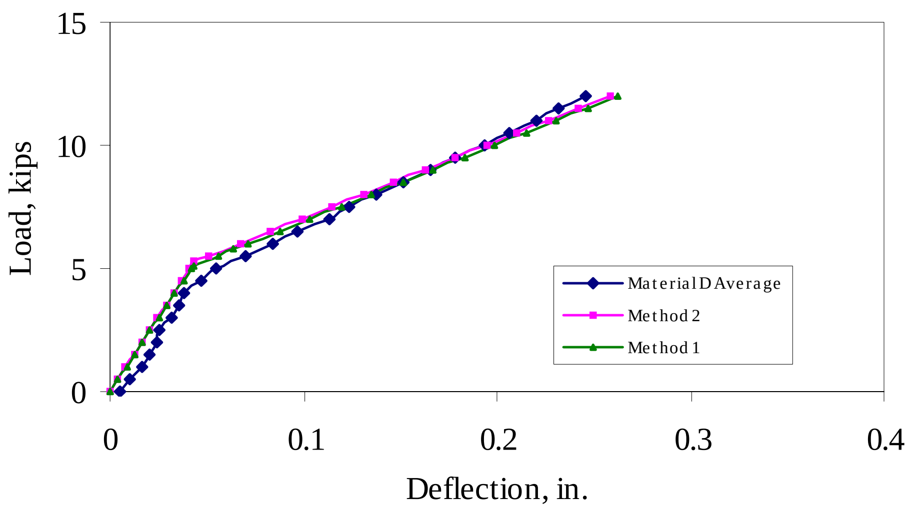


b.) Material B

Figure 4.1 Theoretical load/deflection vs. experimental load/deflection plots for the various repair materials.

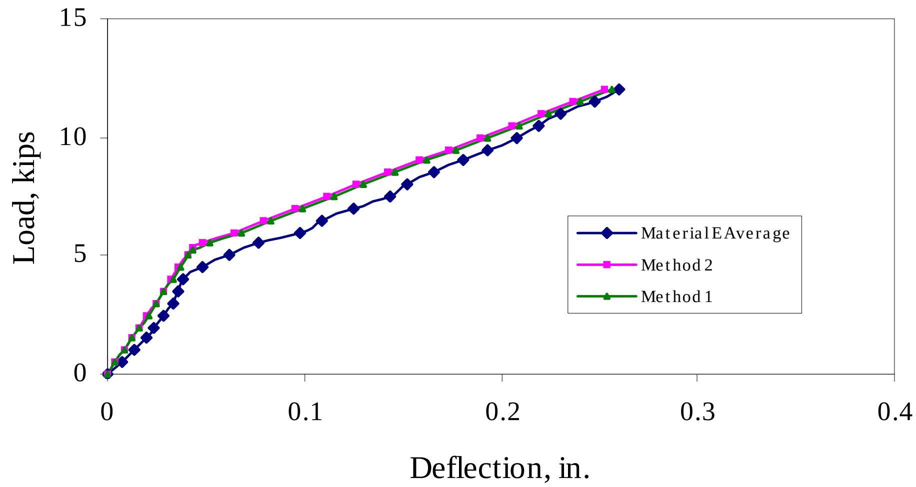


c.) Material C

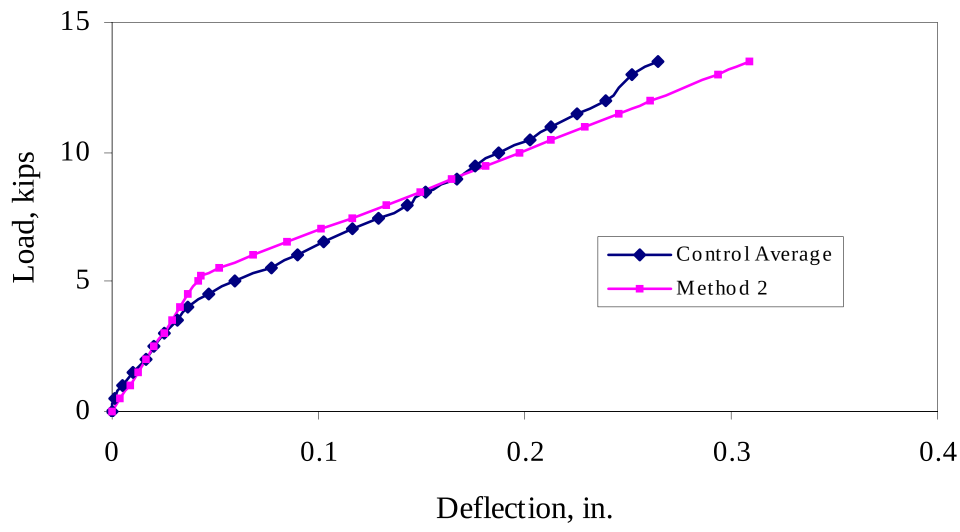


d.) Material D

Figure 4.1 continued.



e.) Material E



f.) Control Beam

Figure 4.1 continued.

Table 4.1 Comparison of moments of inertia determined using Method 1 and Method 2 used to calculate deflections in the repaired beams.

Material	Moment of Inertia (in^4)		Ratio M2/M1
	Method 1 (M1)	Method 2 (M2)	
A	744	862	1.16
B	914	862	0.94
C	887	863	0.97
D	840	858	1.02
E	842	857	1.02

fibers to reach their modulus of rupture, f_r, the beam cracks. When the beam cracks, the moment of inertia decreases due to the loss of effective section, and the beam is not as stiff. Therefore, the slope of the load/deflection plot decreases after the beam has cracked. The information shown in the load/deflection plots was used to predict the behavior of beams repaired with different repair materials. Both methods, Method 1 and Method 2, seem to do well at predicting the cracking load. When the modulus of elasticity of the repair material is close to the modulus of elasticity of the concrete, the gross moment of inertia is very similar in the two methods. The cracking loads are also very close when the repair material and the concrete have similar moduli of elasticity.

The moments of inertia based on Method 1 and Method 2 respectively, theoretical ultimate load, theoretical cracking load, and theoretical deflection at the cracking load, are presented in Appendix A. The ultimate load for each beam with two point loads applied at each third-point, based on strength analysis, is 6.7 kips. This corresponds to a service level load of 3.9 kips using the American Concrete Institute (ACI) live load factor of 1.7. The load/deflection plots presented in Figure 4.1 show the total load applied to each beam, or twice the load at the third-points; which means that the beams all exceeded the predicted calculated load. Therefore, adding any repair material stiffens a damaged beam.

4.1.2 Specific Material Behavior

Material A

Shown in Figure 4.2 is a damaged beam repaired with Material A. Material A had the lowest compressive strength (4,400 psi) and modulus of elasticity (2x106 psi) of any of the materials tested. The theoretical cracking load determined using Method 1 was close to the average experimental value; the theoretical cracking load determined using Method 2 was higher. Even though there is a difference in the cracking loads, the deflection of the beam is predicted quite accurately in the uncracked region. Once the beam had cracked, it was stiffer than predicted. Method 2 did a better job predicting the behavior of the beams. This was due to the large difference in the modulus of elasticity of the repair material and that of the concrete. At the ultimate load, the bond between the beam and the Material A failed. Large chunks of the patch fell out at failure.

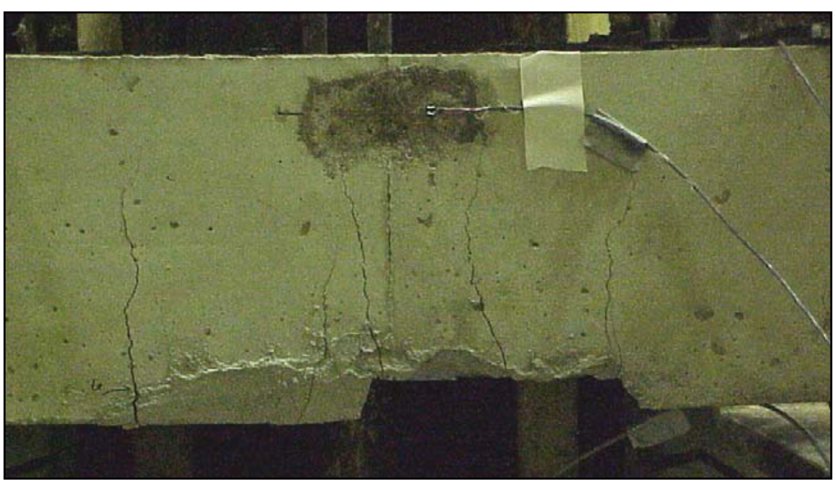


Figure 4.2 Material A damaged patch.

Material B

Material B was the stiffest material, and it also had the highest compressive strength (12,340 psi). The load/deflection behavior (Figure 4.1b) is predicted well by both methods. Even though the repair material has a compressive strength three times that of the concrete, the modulus of elasticity is not as high proportionally. Because the models for load/deflection behavior are not based on compressive strength, but based on modulus of elasticity, they predict beam behavior well. Like for Material A, the cracked beam was stiffer than predicted. Material B remained attached to the beam better at ultimate load than Material A. There were large cracks, some debonding, but no pieces of the patch fell out. The failure was along the bond line and not in the repair material.

Material C

Material C had approximately the same modulus of elasticity as the concrete it was used to repair. Figure 4.3 presents a typical patch failure with Material C. Due to the similarity in the modulus of elasticities of the repair and the damaged concrete beam, deflections calculated using the two methods were nearly identical. Both methods also predict the actual behavior very well. Like other materials, the cracked behavior was stiffer

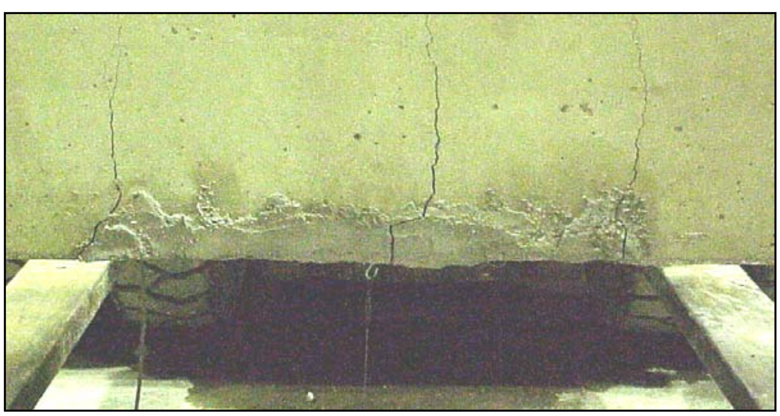


Figure 4.3 Material C patch at ultimate load showing the failure at the patch ends.

than predicted by both methods. Unlike the other repair materials, chunks of Material C fell out at the ends of the patch.

Material D

Again the modulus of elasticity of the repair material was nearly identical to that of the concrete repaired, and the behavior, both cracking load and uncracked stiffness, were predicted very well by both methods. Shown in Figure 4.4 is the complete patch failure. Material D fell off of the beam at a lower load than other repair materials. The failure was along the interface and there was little to no warning. Unfortunately, all Material D patches failed similarly.

Figure 4.4 Complete Material D patch failure.

Material E

Material E has a modulus of elasticity closest to that of the repaired concrete of any of the repair materials. Because of this similarity, Method 1 and Method 2 give results that are nearly identical. However, both methods overestimate the cracking load of the beam, which was common in all repair combinations. The cracking behavior of concrete is difficult to predict due to the heterogeneous nature of the material. However, the cracking load determined by Method 2, is within 10% of the actual value. Material E debonded from the beam at ultimate load easier than the other repair materials. Like many of the Material D patches, the patches fell out of the beam very cleanly. The other repair materials would fall out of the beams after a light tapping with a hammer. The Material E patches would often fall out at a relatively low load. Twice the patch fell out before the reinforcement in the beam yielded. A typical Material E patch failure is shown in Figure 4.5.

Figure 4.5 Material E patch failure along the bond line.

4.2 Beam Specimen Shear Test

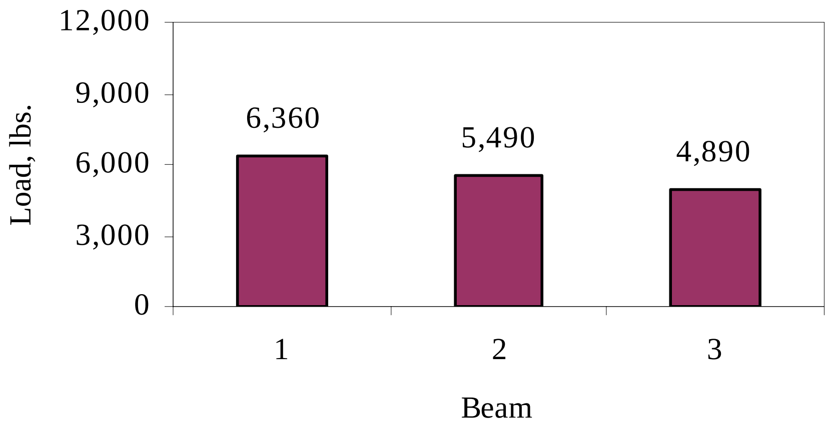
Every patch sheared from the damaged section in the concrete beam very cleanly. In most cases, there was little concrete removed from the beam or very little repair material still attached to the beam. If there was any concrete removed, it tended to be on the bottom of the beam along the line between the beam and the patch. While observing the tests in which the patches had concrete attached at failure, it was very apparent that there was some prying of the patch away from the beam instead of "pure" shear. The prying action of the force created tension stresses at the top of the patch and compressive stresses at the bottom of the patch. Since concrete shear failures are essentially tensile in nature, there was sufficient net compressive stress at the bond interface to eliminate the shear failure at the bond interface and cause a shear failure in the base concrete.

Ultimate failure loads for each material tested in the push out shear test are presented in Figure 4.6; an average ultimate failure load for each material is presented in Figure 4.6f. The three values given for each material (in Figures 4.6a to 4.6e) show the results of the individual push out shear tests.

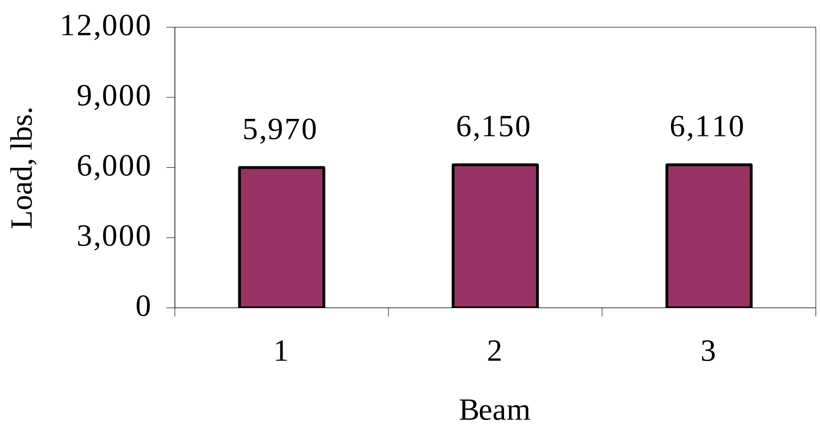
The shear strength of each of the repair materials determined by the test performed in this study are presented in Figure 4.7; it is obvious that all of the repair materials performed much better than the plain concrete. Materials A, C, and D all had very similar shear strength, while Material E had the highest. Material B is not included in this test because the testing frame was damaged during the process of testing the Material B samples. The shear stress presented in Figure 4.7 is calculated by dividing the average shear load in Figure 4.6f by the shear area determined in Chapter 3. Because all of the beams used in the test were constructed with patches cast from the same form, the shear area in each beam is essentially the same. This means that the only variable that changes from test to test is the shear strength of the material. The test used was very different from any test used by manufacturers to report shear strength (see Table 3.1). The test used is a better indication of patch behavior because the test setup is more like the conditions the patch would experience in service if a lateral load accidentally struck it.

4.3 Wedge Cylinder Zero Freeze/Thaw Cycle Test Results

The results of the cylinder specimen tests at zero freeze/thaw cycles, and the control values used in this report are presented in Table 4.2. The first column in Table 4.2 presents the material type, the second column lists the age of the specimen when tested, the third

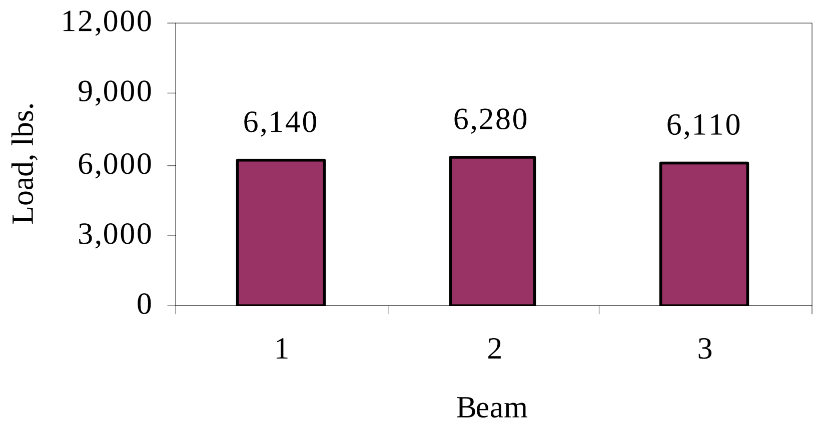


a.) Material A

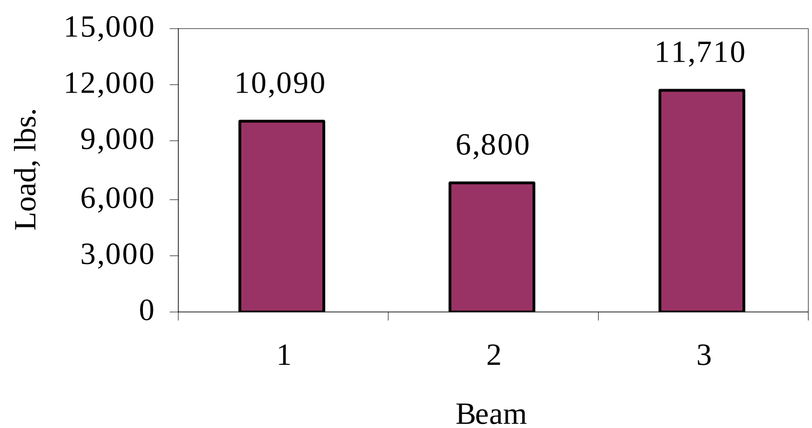


b.) Material C

Figure 4.6 Failure load results for the push out shear tests.

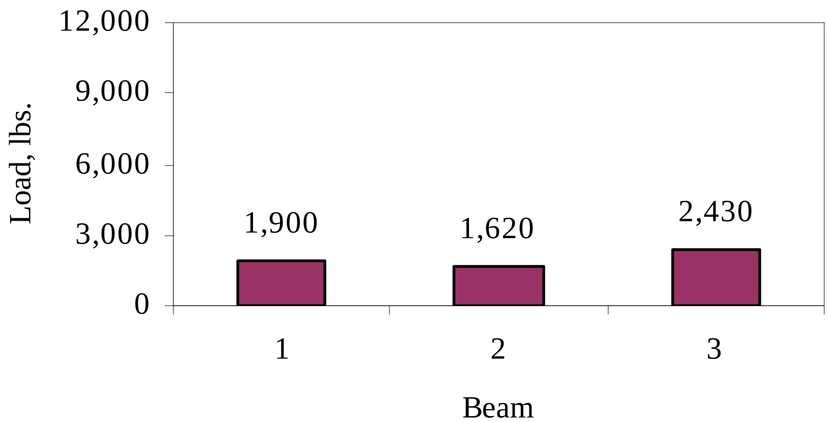


c.) Material D

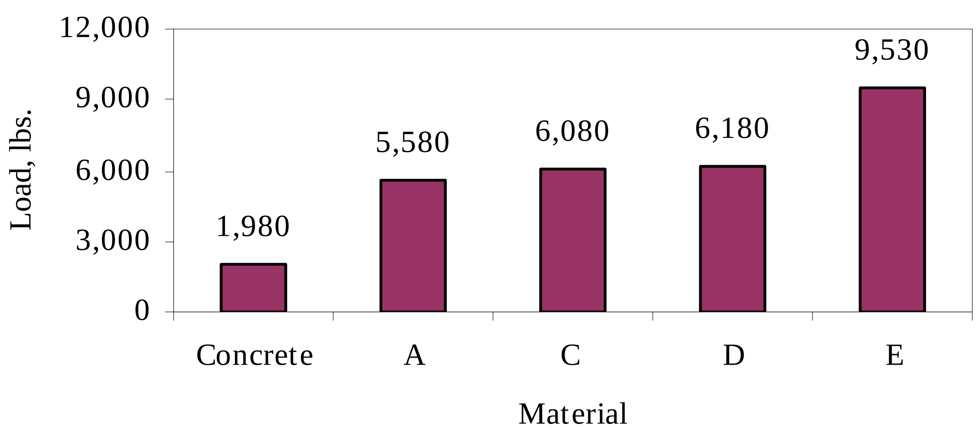


d.) Material E

Figure 4.6 continued.



e.) Plain concrete



f.) Average beam shear load

Figure 4.6 continued.

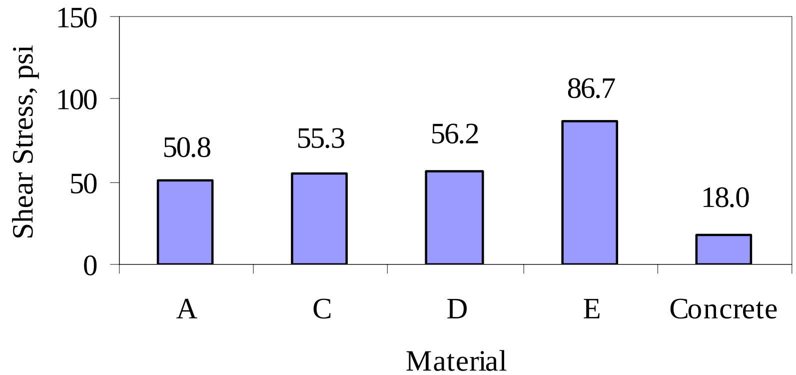


Figure 4.7 Experimentally determined shear strength of beam repairs.

column lists the type of cylinder tested, and the final column presents the average compressive failure load. Full cylinders were made entirely of the various repair materials, and shear cylinders were a combination of repair material and concrete as described in Chapter 3. All of the full cylinders, except for Material B cylinders, failed very explosively. The Material B samples failed in a ductile manner for a cementitious material. The load rate decreased and was terminated once it was determined that there was a failure in the material. The values for the full cylinders were compared to the manufacturers' published data in Table 3.1. Full cylinder samples failed in compression only and the wedge cylinders failed in shear only, unless noted otherwise in Table 4.2. The experimentally determined material properties were slightly different than the reported values from each manufacturer due to laboratory curing conditions. The largest difference in performance between the reported material properties and the experimentally determined material properties (Table 4.3) occurred in Material D. The experimental compressive strength values for each Material D sample were very close together (6,200 - 6,590 psi), which leads to very good confidence in

Table 4.2 Summary of experimental wedge cylinder failures.

Material Type	Repair Material Age (days)	Type of Cylinder	Average Failure Load (lbs)
A	38	Full	31108 (C)
A	38	Shear	27570 (S)
B	31	Full	87670 (C)
B	31	Shear	33,940 (C) 34,780 (S)
C	23	Full	42420 (C)
C	23	Shear	38,670 (C) 35,070 (S)
D	15	Full	44970 (C)
D	15	Shear	69140 (S)
E	14	Full	53870 (C)
E	14	Shear	40720 (S)

(C) denotes compressive failure
(S) denotes shear failure

Table 4.3 Comparison of experimentally determined compressive strengths and reported compressive strengths.

Material	Experimental Coimpressive Strengths (psi)	Reported Compressive Strengths (psi)	% Difference
A	4,400	5,000	-12.0
B	12,400	11,000	12.7
C	6,000	7,000	-14.3
D	6,360	5,000	27.2
E	7,620	7,400	3.0

the experimental values. Because of the difference in the reported and experimental values, the experimental values will be used for the calculations in this report. The most important information in the zero cycle cylinder data is the strength of the wedge samples. With no exposure to freeze/thaw cycles, most of the cylinders failed in compression, with a few exceptions. However, all of the Material E cylinders and one of the Material B cylinders failed in shear. The one Material B sample that failed in shear was not properly consolidated into the bottom of the wedge. When the repair material was not properly consolidated into the bottom of the wedge, the effective shear area was decreased. Because the shear area of this sample was less than that of the other samples used, it had less shear capacity. Even though this sample was slightly different than the other samples, the information gathered from this sample was still used in the study. Due to the small defect in this particular sample, it is justifiable to use it in this study.

4.4 Wedge Cylinder 110 Freeze/Thaw Cycle Results

Table 4.4 presents the results of the 110 freeze/thaw cycle test results, while Figures 4.8 and 4.9 show typical freeze/thaw specimen compressive failures and shear failures, respectively. The modulus of elasticity and coefficient of thermal expansion values shown in Table 4.4 were reported by the manufacturers. The experimental information reported is in terms of failure load and failure stress. The failure load is useful because it is independent of failure mode. Failure stress is important because each manufacturer reports bond stress in their product information.

Material A

All of the cylinders used in the 110-freeze/thaw cycle tests are the wedge type cylinders. Shown in Figure 4.10 are the failure loads for the six Material A cylinders. The

Table 4.4 Wedge cylinder compressive test results after 110-freeze/thaw cycles.

Material Type	Failure Load (lbs.)	Repair Material Age (days)	Failure Stress (psi)	Slant Shear Bond Strength [ASTM 882] (psi)	Repair Modulus of Elasticity (psi)	Repair Coefficient of Thermal Expansion (in./in.°F)
A	29,460	48	2,080	1,500	2.00×10^{6}	5.7×10^{-6}
	13,860	48	980			
	23,920	48	3,385			
	25,160	49	3,560			
	31,440	49	2,225			
	27,430	49	3,880			
B	24,300	41	3,440	3,000	5.00×10^{6}	6.3×10^{-6}
	33,620	41	4,755			
	26,110	41	3,695			
	18,310	42	1,295			
	35,650	42	5,040			
	31,150	42	4,410			
C	33,450	35	2,370	2,200	4.37×10^{6}	4.2×10^{-6}
	23,260	35	1,645			
	33,500	35	2,370			
	25,600	36	1,810			
	27,690	36	1,960			
	32,500	36	2,300			
D	25,750	25	3,640	1,000	4.10×10^{6}	4.5×10^{-6}
	36,390	25	5,150			
	29,650	25	4,190			
	33,430	26	4,730			
	28,840	26	4,080			
	34,470	26	4,880			
E	36,570	24	2,590	2,680	4.22×10^{6}	8.0×10^{-6}
	37,140	24	2,630			
	36,820	24	2,605			
	29,960	25	2,120			
	34,280	25	2,425			
	31,920	25	2,257			

Figure 4.8 Typical compressive wedge cylinder failure.

Figure 4.9 Typical shear wedge cylinder failure.

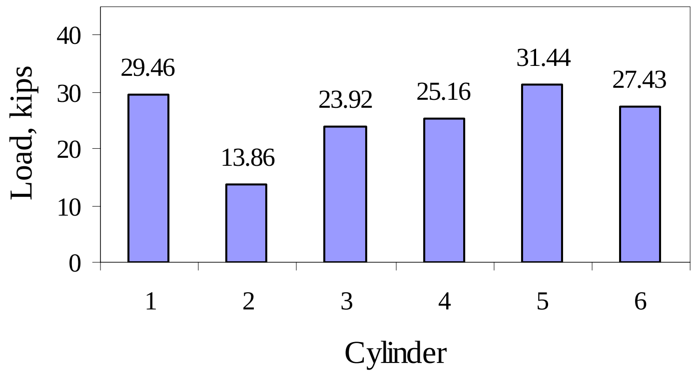


Figure 4.10 Failure load of Material A wedge cylinders subjected to 110-freeze/thaw cycles.

Material A control sample wedges all failed in compression, while 3 of the 6 110-freeze/thaw cycle wedges failed in compression and the other three failed along the bond surface. This is an important observation because the bond strength of repair materials is a major concern of the overall effectiveness of the repair. The coefficient of thermal expansion of Material A is higher than the concrete by 25%, which will cause detrimental movement of the two materials when exposed to freeze/thaw cycles. The relative movement of the two materials caused localized stresses at the bond interface and decreased the strength of the repair system. Cylinder 2 failed in shear at a significantly lower load than the other samples, but within 50% of the maximum of the other cylinders. It was decided that this low load was not too low to be used in this study. This was not due to testing error, so the results have been included in the material average. The other five samples failed at consistent loads.

Material B

Figure 4.11 presents the failure loads of the six 110-freeze/thaw Material B cylinders. With zero freeze/thaw cycles the wedge cylinders failed both in compression and in shear, but after the freeze/thaw cycles, the majority failed in compression. The exception was

Cylinder 4, which failed in shear at a lower load than the other cylinders. The cylinders that failed in compression failed in the base concrete rather than in the repair material. The base concrete was very brittle and was easy to pick apart by hand because of the freeze/thaw cycles. Unfortunately, it is difficult to comment on the performance of Material B with freeze/thaw cycles due to the deterioration of the base concrete. The only conclusion that could be made was that the repair material was more durable than the base concrete. All that can be asked of the repair material is to perform as well as the concrete that it repairs.

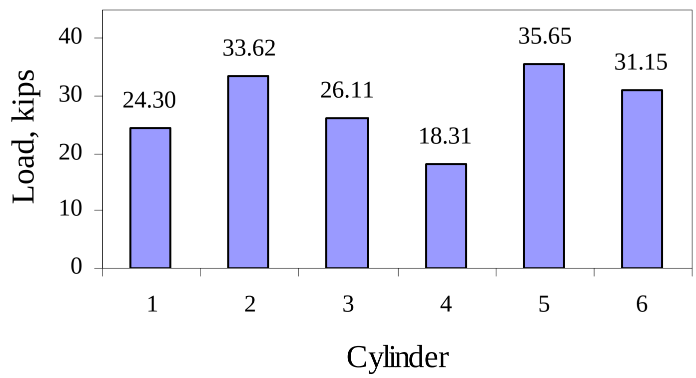


Figure 4.11 Failure load of Material B wedge cylinders subjected to 110-freeze/thaw cycles.

Material C

Figure 4.12 presents the failure loads for the six 110-freeze/thaw Material C cylinders. All of the 110-freeze/thaw cycle samples failed in shear along the bond plane and at reasonably similar loads. The material has a coefficient of thermal expansion similar to the base concrete, which minimizes stresses along the bond plane during freeze/thaw cycles.

Material D

Figure 4.13 presents the results of the six 110-freeze/thaw Material D cylinders. All of the wedges in the 110-freeze/thaw cycle tests, like the zero-freeze/thaw cycle tests, failed in compression. Some failures occurred in the base concrete while others occurred in the repair material, but no failures occurred along the bond line. This observation is interesting because according to the reported manufacturers' information, the bond strength of Material D is the lowest of any of the materials used in this investigation.

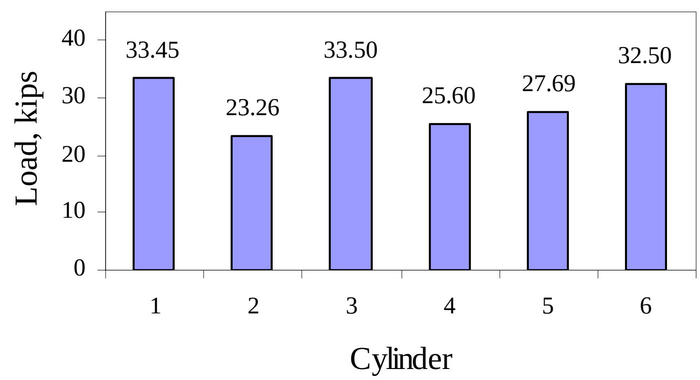


Figure 4.12 Failure load of Material C wedge cylinders subjected to 110-freeze/thaw cycles.

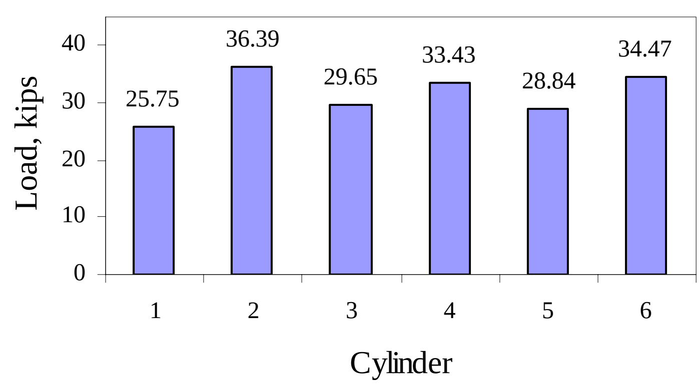


Figure 4.13 Failure load of Material D wedge cylinders subjected to 110-freeze/thaw cycles.

Material E

Figure 4.14 presents the results of the six 110-freeze/thaw Material E cylinders, and shows that the failure loads are very similar. Every sample failed cleanly in shear along the bond line with essentially no repair material remaining on the wedge after failure. There was no deterioration in the base concrete, so all of the decrease in cylinder strength from the zero freeze/thaw cycle cylinders to the 110-freeze/thaw cylinders was due to a decrease in the bond strength.

Shown in Figure 4.15 is the decrease in average cylinder strengths for the various repair materials due to the freeze/thaw cycles. This figure does not indicate the type of failure in the cylinders (i.e. compression or shear). From the discussion of each material, it was apparent that freeze/thaw cycles did not change the failure mode in a manner that affects the average values, so both the shear and compressive failures are included in the average failure values presented in Figure 4.15. Deterioration of the base concrete was more of a factor than failure mode. Base concrete deterioration was a particular concern with Materials B and D. It is apparent from Table 4.5 that Material A performed the best when subjected to freeze/thaw cycles and Material C performed the worst. The percentage decreases are all relative to the initial experimental strength of the individual repair materials.

4.5 Discussion of Test Results

4.5.1 Flexural Test

In Table 4.6, a relative ranking has been assigned to the experimentally determined material properties evaluated in this study. The rating applied to each material in the bond strength column is based purely on the experimentally determined bond strength values, and is not related to the durability of the material. The application rating column in Table 4.6 is a

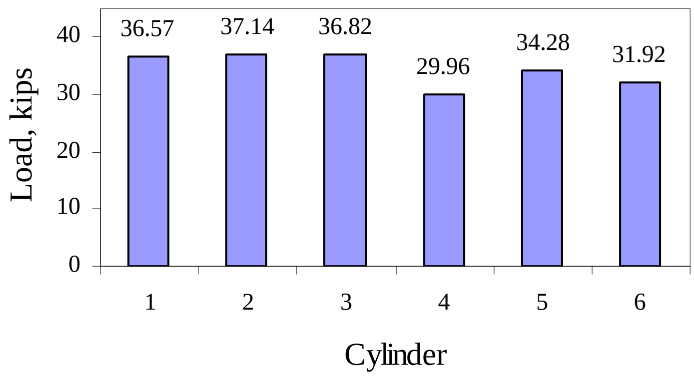


Figure 4.14 Failure load of Material E wedge cylinders subjected to 110-freeze/thaw cycles.

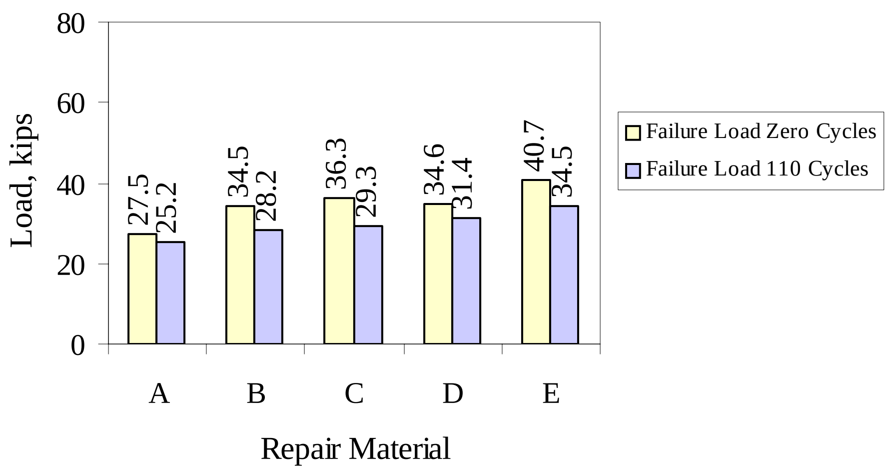


Figure 4.15 Comparison of average wedge cylinder failure loads initially and after freeze/thaw cycles.

Table 4.5 Decrease in wedge cylinder strength due to 110-freeze/thaw cycles.

Material	% Decrease
A	8.40
B	18.20
C	19.20
D	9.10
E	15.40

Table 4.6 Experimentally determined ranking of repair materials based on all tests performed in this study.

Material	Compressive Strength	Bond Strength Zero f/t Cycles	Bond Strength 110 f/t Cycles	Application Rating	Push Out Shear Test
A	5	5	5	5	4
B	1	4	4	2	N/A
C	4	2	3	3	3
D	3	3	2	1	2
E	2	1	1	4	1

qualitative rating assigned to each material based on how easy the material was to apply. As reported in Chapter 3, Material D was the easiest to apply, therefore it receives a ranking of "1". The final column reports the performance of the repair materials in the push out shear test. There was a problem with the Material B samples during this test; therefore the results from this test are not applicable.

Compressive strength and bond strength respectively, are not solely responsible for the durability of a repair material and any repair based solely on these two properties will most likely have long term problems. Each of these two properties needs to be considered when a repair material is being selected, but it could be hazardous to select a material base solely on one of these two properties. For example, Table 4.6 indicates that Material B ranks poorly in bond strength determined from the slant shear tests with zero freeze/thaw and 110-freeze/thaw cycles. This indicates Material B will not bond well to the base concrete when freeze/thaw cycles are applied, and would not be a good material to use in Iowa. Following sections of this thesis evaluate the performance of the different repair materials considering the specific properties that affect the performance of the material.

Load/deflection behavior was discussed in the Beam Results section of this report and a method for modeling a repaired beam was proposed. Table 4.7 shows the results of the theoretical cracking load calculation versus the actual cracking load.

Theoretical cracking loads shown in Table 4.7 are based on the two material beam (Method 2). Relative scores listed in the last column are based on the effectiveness of the theoretical model to predict the cracking load of the beam. The ability to predict the cracking load is an important consideration for engineers when repairing a concrete beam. When a beam cracks, water and contaminants are able to migrate into the beam. If the water contains chlorides, corrosion of the reinforcement can be accelerated beyond that of an uncracked beam, which leads to loss of reinforcement area and debonding of the reinforcement. Even if the water is not corrosive, it still poses a problem for the beam. If the beam is in a cold climate, the water can freeze. When the water freezes, it expands and can cause serious damage to the concrete.

Table 4.7 Repair material rankings based on predicted flexural cracking load.

Material	Experimental Average Cracking Load (kips)	Theoretical Cracking Load (kips)	P_{exp}/P_{theo}	Ranking
A	3.92	4.80	0.82	4
B	4.50	4.80	0.94	1
C	2.99	4.72	0.63	5
D	4.50	5.25	0.86	2
E	4.50	5.34	0.84	3

Cracking load calculations, based on flexural stresses, are shown in Appendix A and discussed previously in this chapter. Load/deflection behavior is a function of the depth of the neutral axis, the modulus of rupture of the concrete, and the gross moment of inertia of the repaired beam. Both the gross moment of inertia and the depth of the neutral axis are a function of the modular ratio (E_r/E_c); where E_r is the modulus of elasticity of the repair material, and E_c is the modulus of elasticity of the concrete. In order to understand why each beam/repair material system behaves the way it does, it is useful to look at the relative ratios of the modulus of elasticity of the repair materials and base concrete. Table 4.7 shows that the experimentally determined cracking load for Material B is closest to the predicted value and the experimentally determined cracking load for Material C is the farthest from the predicted cracking load. This comparison is valid because the theoretical analysis used to determine the cracking load was very close to the experimental cracking load as seen in Figure 4.1a-e. If the only variable used to predict the cracking load is the ratio of the modulus of elasticity of the repair material to the modulus of elasticity of the base concrete (Table 4.8), it makes sense that the prediction of the cracking load is related to the modular ratio. Table 4.8 (pour 3 was only used for the control beams) also indicates that the modular ratio for Material B is the highest (1.24) and the modular ratio for Material A is the lowest (.50). As the modular ratio decreases so does the ability of Method 2 to accurately predict the cracking load. The exception is Material C. It cracks at a load much lower than predicted. This was because there were several shrinkage cracks in the repair material before the flexural tests started. These shrinkage cracks decrease the structural depth of the beam, so the actual structural depth of the beam is less than the depth assumed in the model. When the structural depth is smaller than that assumed in the theoretical model, the experimental data are skewed. The beam is less stiff (more deflection with equal load) and cracks sooner. This explanation is validated by Figure 4.1c. The slope of the load/deflection curve of Material C is less than the slope predicted and the cracking load is much less than predicted.

Table 4.8 Summary of ratio of modulus of elasticity of repair material to the modulus of elasticity of the base concrete.

Material	Material, E_r 10^6 (psi)	Concrete Pour	Concrete, E_c 10^6 (psi)	E_r/E_c
A	2.00	1	4.03	0.50
B	5.00	1	4.03	1.24
C	4.37	2	3.95	1.11
D	4.10	5	4.52	0.91
E	4.22	4	4.45	0.95

The other materials follow the trend that varies with the modular ratio. One assumption in this method for predicting the cracking load is that the concrete cracks before the repair material cracks. Based purely on the simple ACI equation (f_r = modulus of rupture in psi, f'_c = 28 day compressive strength in psi):

$$f_r = 7.5\sqrt{f_c'}$$

the modulus of rupture of the repair materials would be higher than the concrete since many of the repair materials have additives that increase the tensile strength of the material, which are not included in the ACI modulus of rupture relation. Other important data taken from the flexural tests are the load/deflection curves of the repaired beams. Of particular interest is the portion of the behavior of the beam in the uncracked region. Once the beam cracks, the portion of the concrete in tension is obviously no longer effective structurally. In the tests performed in this study, the patch was in the tension portion of the beam. Therefore, it does not affect the behavior of the beams once the beam cracks. The best way to evaluate the performance of the repair materials with respect to load/deflection behavior is to relate the slope of the experimental load/deflection curves to the predicted load/deflection slopes (see

Table 4.9). It does not make sense to compare the stiffness of beams repaired with different repair materials because the base concrete does not have the same strength. The theoretical slopes of the load/deflection plots are based on theoretical behavior of a beam of two materials (Method 2). Slopes are calculated by dividing the cracking load by the deflection at the cracking load. Rankings are assigned to each material based on the ability of the theoretical method to predict actual behavior. Material B's load/deflection behavior in the uncracked region of the load/deflection behavior is closest to the expected behavior, and was ranked "1". Material A was ranked "5" because its actual slope is the furthest from the predicted slope.

Table 4.9 Comparison of experimental and theoretical uncracked load/deflection slopes.

Material	Experimental Slope, S_{exp} (lbs/in)	Theoretical Slope, S_{theo} (lbs/in)	S_{exp}/S_{theo}	Rank
A	74,070	110,680	0.669	5
B	98,180	110,680	0.887	1
C	79,040	108,565	0.728	4
D	94,880	121,570	0.78	2
E	94,120	123,430	0.763	3

Load/deflection predictions were made based on elastic beam behavior of a beam loaded at the one-third points. The general equation for the deflection as a function of load for each repair material is shown in Appendix A. The loads were applied at the same points for all of the beams (32 in. from each end), which leaves the modulus of elasticity and the moment of inertia as the only relevant variables from beam to beam. In order to make a conclusion about the stiffness of the beams; the modulus of elasticity of the repair material needs to be examined (Table 4.8). Again the material that performs the best is the stiffest material, Material B. The material that performs the worst is Material A, which has the lowest modulus of elasticity. The other three materials have similar modulus of elasticities. The deflection of reinforced concrete beams is always difficult to predict due to the brittle, heterogeneous nature of the material. All of the actual slopes are less than the predicted theoretical slopes, which means that the beams are less stiff than theoretically predicted. A material with a higher modulus of elasticity would behave more ideally because stiffness is directly proportional to the modulus of elasticity.

In addition to structural behavior of repaired concrete beams, engineers are often concerned with the ability of the patch to bond to the beam. Pieces of repair material that fall onto a roadway may cause serious injury to vehicles and their occupants which are driving underneath them. The final information (Table 4.10) that was obtained from the flexural tests was the ability of the patch to adhere to the damaged concrete at the ultimate load of the beam. There is no way to quantify the bond between the patch and the beam at the beam's ultimate load because some portion of the patch had debonded; therefore the rankings are qualitative from the notes taken while the beams were tested. Material E deteriorated more than any of the other materials at a high load. The patches completely debonded before the beam had yielded. Failure occurred along the bond line because there was little to no cracking in the patch itself. This suggests that the bond was very weak. On the manufacturer's instructions for application, moist curing is listed as required. This step was not followed because moist curing is not used very often in the field and was not used on the other materials. Material E was also poured very wet (almost infinite slump). When all of the water evaporates, there is significant potential for shrinkage.

Table 4.10 Relative rankings of repair material's bond strength in the flexural test.

Material	Ranking
A	3
B	1
C	2
D	4
E	5

The combination of high slump and lack of moist curing was most detrimental to the bond strength of Material E. The other material that performed poorly was Material D. Material D fell out almost as easily as Material E, but not in the same manner. It fell out in chunks from the middle, while the edges remained bonded to the beam. Material D was the easiest material to apply because it was possible to apply the repair in one lift. Even though the material was able to support itself in one lift, it seems that there was a lack of bond strength at the center. The other materials performed adequately.

4.5.2 Push Out Shear Test

The push out shear test was performed to evaluate the ability of the repair materials to remain bonded to the beam after a vertical load had been applied. The ranking of the materials from these tests are shown in Table 4.11. The results listed in Table 4.11 show how all of the materials performed relative to the other materials. Even though Materials A, C, and D are ranked 4, 2, and 1, respectively, it would not have been unreasonable to give them a ranking of 2 because Materials A, C, and D all performed very similarly. Figure 4.7 presented the experimental shear strength of all of the materials, and it can be seen that Materials A, C, and D showed nearly identical shear strengths. Material E performed the best by far. Most importantly, the plain concrete samples did not perform well at all. All of the plain concrete specimens were poured very stiff and were not wet cured. This lead to large shrinkage cracks, and because of the large amount of shrinkage the bond between the base concrete and the patch was not very strong. All of these materials failed at a shear stress that is much lower than any reported value listed by any manufacturer. Possible causes of failure were the flexural shear stresses that occurred before the pure shear load and the poor bond due to over head application. This test is not analogous to any published test, but is a valid concern to repair engineers. There is no way to predict the pure shear strength of a repair patch based on information provided by manufacturers.

Table 4.11 Repair material ranking based on the push out shear test results.

Material	Ranking
A	4
B	N/A
C	3
D	2
E	1
Concrete	5

4.5.3 Bond Strength

Table 4.12 presents the relative rank of the bond strength of each repair material without freeze/thaw cycles. The two columns show slightly different results between the ranking based on the manufacturers' reported bond strengths and the experimental bond strengths. The difference between the reported bond strengths and the experimental bond strengths is caused by the different tests used to report bond strength. This study evaluated bond strength using a modified ASTM 882 test; which was explained in Chapter 3. As stated in Chapter 3, in the author's opinion the modified ASTM 882 adequately measures the bond strength between a repair patch and a damaged concrete surface because the surface used in the modified test is roughened.

Table 4.12 Comparison of experimental rankings and manufacturers' reported material property ranking of bond strength of repair materials with zero freeze/thaw cycles.

Material	Experimental Bond Strength Ranking	Manufacturers' Reported Bond Strength Ranking
A	5	4
B	4	1
C	2	3
D	3	5
E	1	2

4.5.4 Bond Strength with Freeze/Thaw cycles

The bond strength information that is useful for repair material selection is the decrease in strength due to freeze/thaw cycles. Table 4.13 presents the ranking of bond strength after 110-freeze/thaw cycles and the percentage decrease of the wedge cylinders from the zero freeze/thaw tests. The second column in Table 4.13 shows the ranking given to the repair materials based on the absolute value of the capacity of the cylinders. The third column shows the percentage decrease of the wedge cylinders from the zero cycle to the 110-freeze/thaw cycle tests. The final column gives a ranking based on the durability of each material. Rankings are based on the percentage decrease of the bond strength of each material, not the absolute value of the bond strength. Throughout this thesis, it has been stressed that repair patch durability should be the ultimate concern of the designing engineer. The last column of Table 4.13 lists a key statistic for determining patch durability. The material property that most directly influences patch durability due to freeze/thaw cycles is the coefficient of thermal expansion. It has been stated earlier that incompatibility of the repair material and in-place concrete with respect to thermal movement can cause large internal stresses. Also, materials with a larger coefficient of thermal expansion do not perform as well as materials with a lower coefficient of thermal expansion with the same number of freeze/thaw cycles. The ratio of the coefficient of thermal expansions of each repair material to the coefficient of thermal expansion of the concrete it was used to repair are presented in Table 4.14. The next to last column shows the ratio of the two values. The ratio of the two coefficients of thermal expansions is an indication of compatibility; the closer the ratio is to one, the more compatible the repair material is thermally. It can be seen that Material E is the least compatible because it has the highest ratio, and Material C is the most compatible because it has the ratio closest to one. It makes no difference if the coefficient of thermal expansion of the repair material or base concrete is the higher of the two.

Table 4.13 Ranking of bond strengths subjected to freeze/thaw cycles.

Material	Bond Strength Ranking with 110 freeze/thaw Cycles	% Decrease from Zero freeze/thaw Cycles	Durability Ranking
A	5	8.4	1
B	4	18.2	4
C	3	19.2	5
D	2	9.1	2
E	1	15.4	3

Table 4.14 supports the claim that freeze/thaw cycles affect repair material durability. However, there was no discernable trend in the data collected in the cylinder test based on the relative value of coefficient of thermal expansion of the repair material and that of the beam. This is due to the fact that the cylinders were unconfined. If the materials were tested in a freeze/thaw test where the materials were confined, like the conditions a patch material

would experience in a beam without feathered edges, the results would be very different. If a patch material with a large coefficient of thermal expansion was used to repair a beam with a low coefficient of thermal expansion, the patch would tend to move more due to temperature changes than the beam. If the temperature decreased, this would lead to tensile stresses at the patch edges and compressive stresses in the concrete beam along the vertical surface. The opposite stresses would occur when the temperature decreased or the relative ratio of the coefficient of thermal expansion in the patch and in the beam was reversed.

Table 4.14 Comparison of the coefficient of thermal expansion of repair material to the coefficient of thermal expansion of the repaired concrete.

Material	Coefficient of Thermal Expansion of the Repair Material (10^{-6}in/in^{o}F)	Coefficient of Thermal Expansion of Concrete (10^{-6}in/in^{o}F)	Ratio of Repair to Concrete	Decrease in Wedge Capacity (%)
A	5.7	4.5	1.27	8.4
B	6.3	4.5	1.40	18.2
C	4.2	4.5	0.93	19.2
D	5.4	4.5	1.20	9.1
E	8.0	4.5	1.78	15.4

5. SUMMARY AND CONCLUSIONS

5.1 Summary

Several reports have been published (1-19) that address the topic of concrete repair, but few have taken the next step and suggested a method for selecting a repair material. This study attempted to determine the repair material properties that are the most important for durable concrete repairs and then suggest a method that repair engineers can use to select a repair material.

In order to select a material to repair concrete, an engineer must be aware of two factors: 1) the repair material's compatibility with the existing concrete, and 2) the application of the repair material. In general, manufacturers report both of these topics for most materials in sales catalogs. Usually, the reported material properties in sales catalogs are compressive strength, bond strength, and set times. Compressive strength and set times are standard tests that do not vary from manufacturer to manufacturer. Unfortunately, there are several ASTM standards that can be used for evaluating bond strength. The most common is ASTM 882 because it is an easy test to perform and to understand. When comparing the bond strength of different materials, it is important to understand the different tests and not be misled by any large value of bond strength.

Five repair materials, with a variety of material properties and recommended by their manufacturers for overhead use, were selected for this study. All of these materials were cementitious materials with different chemical additives that enhanced f/t resistance, increased bond strength, and/or decreased the material weight in comparison to Portland concrete.

Beam specimens were constructed and then damaged to simulate a horizontal impact load. These beams were repaired and then tested initially with a vertical load and then a horizontal load to determine the strength of each of the repair materials in different loading conditions. Cylinder specimens were also constructed to determine the pure compressive strength and the shear bond strength of each repair material. Using a wedge test similar to ASTM 882, the wedge cylinders were subjected to freeze/thaw cycles and an axial load to test the shear bond strength and the durability of the bond.

The load/deflection behavior, cracking load, patch bond, and shear strength of all of the different concrete/repair material combinations were analyzed to determine if there were any trends in the data. It was determined that modulus of elasticity was the most influential material property. The load/deflection behavior of beams repaired with materials a higher modulus of elasticity than the base concrete were stiffer up to the cracking load and were in better agreement with a simplified theoretical model.

Several different methods were used in this study to measure repair material bond. The push out shear test measured the pure shear strength of the material. All of the repair materials failed at a much lower stress than predicted by any of the published shear strengths. However, all materials performed much better than plain concrete. Due to the poor performance of the repair materials compared to the published values, it was not possible to make any correlation between published strengths and the actual strength of the materials. Wedge cylinders were also used to evaluate the bond strength of the repair materials. The wedge cylinder samples were exposed to 110 freeze/thaw cycles and then axially loaded to failure. The materials that performed the best had a coefficient of thermal expansion similar to the base concrete.

5.2 Recommendations

The purpose of this report is to help engineers design effective concrete repairs for bridges or any other situation where concrete is damaged. The preceding material has discussed the general theory behind concrete patching and has shown laboratory tests that proves the importance of several key repair material properties. The following section will show how to select a repair material based on the requirements of the structural system.

5.2.1 Selection Algorithm

It has been shown that the key criteria are modulus of elasticity, bond strength, thermal compatibility, material application and workability, and compressive strength. Of all of these material properties modulus of elasticity and bond strength are the most important. Any selection algorithm should weight these properties most heavily. The algorithm described below and shown in Table 5.1 can be used to compare any prospective repair materials.

1.) Select several repair materials.

2.) Make a table of the modulus of elasticity, bond strength, compressive strength, thermal compatibility (use base concrete coefficient of thermal expansion of 4.5×10^{-6} in./in.°F), and application (based on the specific conditions required).

3.) Make a relative ranking of each material in the five categories listed in Item 2.

4.) Weigh the importance of the modulus of elasticity and the bond strength factors. A weighting factor of two is suggested.

5.) Total the rankings.

Table 5.1 Material ranks based on the proposed material selection algorithm.

Material	Modulus of Elasticity (x2)	Bond Strength (x2)	Coefficient of Thermal Expansion ratio	Compressive Strength	Application	Total Ranking
A	10	8	3	4	5	30
B	8	2	4	1	2	17
C	6	6	1	3	3	19
D	4	10	2	4	1	21
E	2	4	5	2	4	17

From Table 5.1, Materials B or E would be the best repair materials and Material A would be the worst because Materials B and E scored the lowest and Material A scored the highest. The magnitude of the ranking number in the last column is not significant. All that is significant is how the materials rank relative to each other. Of course if an engineer were trying to repair a concrete member exposed to different conditions, the selection algorithm could have different weighting factors. For instance, if compressive strength was a more important material property for the given repair situation, it could be given a weight of 2.

5.3 Conclusions

Repair material selection is a difficult problem for any engineer due to the large variety of repair materials available. This study has identified four material properties that need to be investigated for durable repairs. These properties are based solely on the loading conditions used in this study: flexural tests, push out shear tests, and slant shear tests in which the specimens were subjected to freeze/thaw cycles. If a repair material were applied in a different situation, like to repair a concrete column, other material properties would be more important.

<u>Most Essential Material Properties for an Effective Concrete Repair</u>

1.) Modulus of Elasticity

2.) Bond Strength

3.) Coefficient of Thermal Expansion

4.) Compressive Strength

In order to use the information presented in this thesis effectively, it is important to know generally how repair materials behave. The following provides some simple general information about repair material behavior.

1.) Unless the repair specifically requires a large compressive strength, do not select a repair material based simply on compressive strength.

2.) Select a repair material with a similar modulus of elasticity to the concrete that is being repaired.

3.) In general, repair materials with a high coefficient of thermal expansion degrade faster in freeze/thaw cycles than materials with a low coefficient of thermal expansion.

4.) Understand the tests used by manufacturers to reported bond strength because manufacturers use a variety of tests. Some of the tests are even designed by the manufacturer themselves. For example, if bond strength is a significant factor in selecting the repair material, make sure that the test used by the material manufacturers to report the bond strength applies load to the repair system in the same manner that load will be applied to the repair material in the field.

In addition to the material properties of the repair material, an engineer must be aware of the ability of the material to be placed. This study has identified several factors that influence the behavior of material application.

1.) Make sure that the contractor hired to perform the work is familiar with rapid setting materials and concrete repair.

2.) Mix small batches of material until sufficiently familiar with how each repair material behaves. The five different materials used in this thesis behaved very differently in the same laboratory conditions. The weather, especially temperature and humidity, can change the performance and pot life of a repair material significantly.

3.) Realize that material performance listed in sales catalogs is under laboratory conditions. The application of the material is never as easy as is reported.

4.) Once the material is mixed with water or latex, be prepared to apply the material because materials set up very quickly.

5.) Most manufacturers require either moist curing or a curing additive. The curing additive acts like a seal to keep the moisture from evaporating from the repair patch too quickly. However, in most applications, the same objective can be achieved by moist curing the repair by covering the repair patch with wet burlap. The wet burlap keeps the repair material moist during the critical curing time.

APPENDIX A:

MOMENT OF INERTIA CALCULATIONS

Method 1 - Moment of Inertia Calculations:

Material A and Pour No. 1:

E_{steel} = 29,000 ksi

E_{repair} = 2,000 ksi

$E_{concrete}$ = 4,031 ksi

A_s = .4 in^2 (2 - #4 bars)

Beam Dimensions:

b = 6.0 in.

d = 10.25 in.

Cover = 1.5 in.

$$n_{steel} = \frac{E_{steel}}{E_{concrete}} \qquad n_{steel} = 7.19$$

$$n_{repair} = \frac{E_{steel}}{E_{concrete}} \qquad n_{repair} = 0.50$$

Transform all areas into the base concrete:

$A_1 = (b)(d)$ $\qquad$ $A_1 = 61.5$ in^2

$A_2 = n_{steel}\,(A_s)$ $\qquad$ $A_2 = 2.88$ in^2

$A_3 = n_{repair}\,(b)(1.5\text{ in})$ $\qquad$ $A_3 = 4.47$ in^2

Location of the Neutral Axis (before cracking):

$$\bar{x} = \frac{A_1\left(\frac{d}{2}\right) + A_2(d) + A_3(d + .75in.)}{A_1 + A_2 + A_3} \qquad \bar{x} = 5.72\text{ in.}$$

$$I_{concrete} = \frac{d^3 b}{12} + A_1\left(\frac{d}{2} - \bar{x}\right)^2 \qquad I_{concrete} = 560.2\text{ in}^4$$

$$I_{steel} = A_2\left(d - \bar{x}\right)^2 \qquad I_{steel} = 59.0\text{ in}^4$$

$$I_{repair} = A_3\left[\left(d + \frac{1.5in.}{2}\right) - \bar{x}\right]^2 \qquad I_{repair} = 124.5\text{ in}^4$$

Table A.1 Summary of Method 1 moment of inertia calculations.

Variable	Material				
	A	B	C	D	E
E_{repair} (ksi)	2,000	5,000	4,370	4,100	4,220
$E_{concrete}$ (ksi)	4,030	3,950	4,415	4,525	4,450
n_{steel}	7.19	7.19	7.34	6.51	6.41
n_{repair}	0.5	1.24	1.11	0.92	0.93
A_1 (in^2)	61.5	61.5	61.5	61.5	61.5
A_2 (in^2)	2.88	2.88	2.94	2.61	2.56
A_3 (in^2)	4.47	11.16	9.96	8.29	8.4
$\overline{x}$ (in.)	5.72	6.19	6.11	5.98	5.99
$I_{concrete}$ (in^4)	560.2	608	598.6	583.6	584.1
I_{steel} (in^4)	59.1	47.5	50.3	47.5	46.6
I_{repair} (in^4)	124.5	258.4	237.8	208.7	211
I_{total} (in^4)	743.8	913.9	886.6	839.8	841.7

Note: Moment of inertias are calculated at the centerline of the beam.

Method 2 - Moment of Inertia Calculations:

Pour No. 1 (Used with Materials A and B)

E_{steel} = 29,000 ksi

E_{repair} = 4,031 ksi

$E_{concrete}$ = 4,031 ksi

A_s = .4 in^2 (2 - #4 bars)

Beam Dimensions:

b = 6 in.

d = 10.25 in.

Cover = 1.5 in.

$$n_{steel} = \frac{E_{steel}}{E_{concrete}} \qquad n_{steel} = 7.19$$

$$n_{repair} = \frac{E_{repair}}{E_{concrete}} \qquad n_{repair} = 1.00$$

Transform all areas into the base concrete:

$A_1 = (b)(d)$ — $A_1 = 61.5\ in^2$

$A_2 = n_{steel}\,(A_s)$ — $A_2 = 2.88\ in^2$

$A_3 = n_{repair}\,(b)(1.5\ in.)$ — $A_3 = 9.00\ in^2$

Location of the Neutral Axis (before cracking):

$$\bar{x} = \frac{A_1\left(\frac{d}{2}\right) + A_2(d) + A_3(d + .75in)}{A_1 + A_2 + A_3} \qquad \bar{x} = 6.05\,in.$$

$$I_{concrete} = \frac{d^3 b}{12} + A_1\left(\frac{d}{2} - \bar{x}\right)^2 \qquad I_{concrete} = 590.7\ in^4$$

$$I_{steel} = A_2(d - \bar{x})^2 \qquad I_{steel} = 50.8\ in^4$$

$$I_{repair} = A_3\left[\left(d + \frac{1.5in.}{2}\right) - \bar{x}\right]^2 \qquad I_{repair} = 220.8\ in^4$$

$I_{total} = I_{concrete} + I_{steel} + I_{repair}$ — $I_{total} = 862.4\ in^4$

Table A.2 Summary of Method 2 moment of inertia calculations.

Variable	Pour				
	1	2	3	4	5
E_{repair} (ksi)	4030	3950	4415	4525	4450
$E_{concrete}$ (ksi)	4030	3950	4415	4525	4450
n_{steel}	7.19	7.34	6.57	6.41	6.51
n_{repair}	1	1	1	1	1
A_1 (in^2)	61.5	61.5	61.5	61.5	61.5
A_2 (in^2)	2.88	2.94	2.63	2.56	2.61
A_3 (in^2)	9	9	9	9	9
$\overline{x}$ (in.)	6.05	6.05	6.03	6.03	6.03
$I_{concrete}$ (in^4)	590.7	591.1	589.1	588.7	588.9
I_{steel} (in^4)	50.8	51.8	46.7	45.7	46.4
I_{repair} (in^4)	220.8	220.5	222.1	222.4	222.2
I_{total} (in^4)	862.4	863.4	857.9	856.8	857.5

Note: Moment of inertias are calculated at the centerline of the beam.

Summary of Cracking Load Calculations:

Material A

f'_{cr} = 4,400 psi f'_{cr} = compressive strength of the repair material.

f_{rr} = 500 psi f_{rr} = modulus of rupture of the repair material.

Concrete

f'_{cc} = 5,000 psi f'_{cc} = compressive strength of Pour No. 1 concrete.

f_{rc} = 530 psi f_{rc} = modulus of rupture of Pour No. 1 concrete.

Gross Moment of Inertia based on the transformation of all three materials into the base concrete (Method 1):

$I_g = 743.7 \text{ in}^4$

Location of the Neutral axis based on the transformed area (measured from the bottom of the beam):

$y_t = 6.28$ in

Calculation of the cracking moment of the beam: Based on a transformed section:

$$M_{cr} = \frac{f_{rc} I_g}{y_t} \qquad M_{cr} = 5.23 \text{ ft-kip}$$

Cracking Load for two point loading:

For two point loading: $$M = \frac{PL}{3} \qquad L = 96 \text{ in.} = 8 \text{ ft}$$

In this case, P, is half of the total load applied.

Therefore, $$P_{cr} = \frac{3M_{cr}}{4} \quad (M_{cr} \text{ in ft-kips}) \qquad P_{cr} = 3.92 \text{ kip}$$

Deflection at the center-line of a beam due to two point loading:

$a = 2.667$ ft $\qquad E_c = 4{,}031$ ksi $\qquad L = 8$ ft

$$\Delta_{\mathbb{C}_L} = \frac{P_{cr}a}{24E_cI_g}(3L^2 - 4a^2)$$

$$\Delta_{\mathbb{C}_L} = 0.041 \text{ in.}$$

See Figure A1 for the variables used.

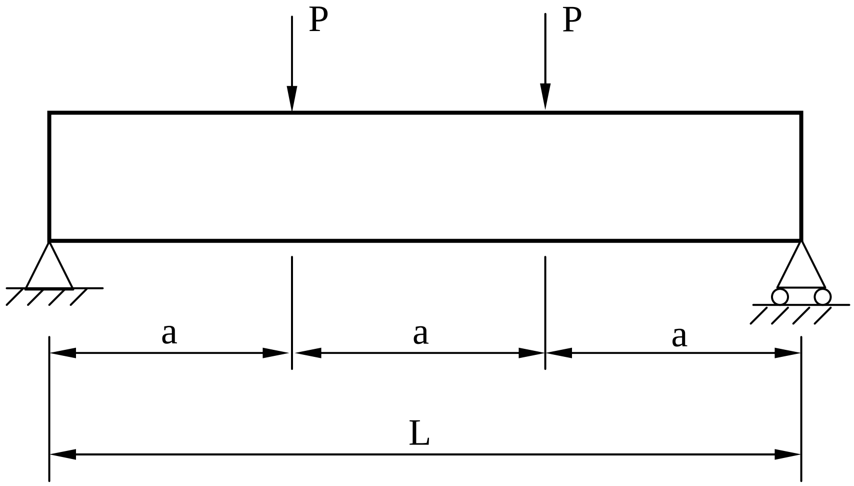


Figure A.1 Loading system used to determine cracking moment.

Table A.3 Summary of Method 1 cracking load calculations.

Variable	Material				
	A	B	C	D	E
f_{cr} (psi)	4,400	12,400	6,000	6,360	7,620
f_{rr} (psi)	500	835	580	600	655
f_{cc} (psi)	5,000	5,000	4,800	6,070	6,300
f_{re} (psi)	530	530	520	585	595
I_g (in^4)	743.8	913.9	886.6	839.8	841.7
y_t (in)	6.28	5.81	5.89	6.02	6.01
M_{cr} (ft-kips)	5.23	6.95	6.53	6.80	6.94
P_{cr} (kips)	3.92	5.21	4.90	5.10	5.21
E (ksi)	4,030	3,950	4,415	4,525	4,450
$\Delta_{℄}$ (in.)	0.041	0.044	0.044	0.043	0.043

Summary of Cracking Load Calculations:

Pour 1

$f'_{cr} = 5{,}000$ psi — f'_{cr} = compressive strength of the repair material.

$f_{rr} = 530$ psi — f_{rr} = modulus of rupture of the repair material.

Concrete — The "repair material" is actually the concrete.

$f'_{cc} = 5{,}000$ psi — f'_{cc} = compressive strength of the concrete.

$f_{rc} = 530$ psi — f_{rc} = modulus of rupture of the concrete.

Gross Moment of Inertia based on the transformation of all three materials into the base concrete (Method 2):

$I_g = 862.4 \text{ in}^4$

Location of the neutral axis based on the transformed area (measured from the bottom of the beam).

$y_t = 5.95$ in.

Calculation of the cracking moment of the beam: Based on a transformed section

$$M_{cr} = \frac{f_{rc} I_g}{y_t} \qquad M_{cr} = 6.40 \text{ ft-kip}$$

Cracking Load for two point loading

For two point loading: $M = \frac{PL}{3}$ $\qquad$ I = 96 in. = 8 ft

In this case, P, is half of the total load applied.

Therefore, $P_{cr} = \frac{3M_{cr}}{4}$ (M_{cr} in ft-kips) $\qquad$ $P_{cr} = 4.80$ kip

Deflection at the center-line of a beam due to two point loading:

a = 2.667 ft $\quad\quad$ E_c = 4,031 ksi $\quad\quad$ L = 8 ft

$$\Delta_{\text{℄}} = \frac{P_{cr} a}{24 E_c I_g}(3L^2 - 4a^2)$$

$$\Delta_{\text{℄}} = 0.043 \text{ in.}$$

See Figure A.1 for the variables used.

Table A.4 Summary of Method 2 cracking load calculations.

Variable	Pour				
	1	2	3	4	5
f_{cr} (psi)	5,000	4,800	6,000	6,300	6,070
f_{rr} (psi)	530	520	580	595	585
f_{cc} (psi)	5,000	4,800	6,000	6,300	6,070
f_{re} (psi)	530	520	580	595	585
I_g (in^4)	862.4	863.4	857.9	856.8	857.5
y_t (in.)	5.95	5.95	5.97	5.97	5.97
M_{cr} (ft-kips)	6.4	6.3	7.0	7.1	7.0
P_{cr} (kips)	4.8	4.7	5.2	5.3	5.3
E (ksi)	4,030	3,950	4,415	4,525	4,450
$_{\text{℄}}\Delta$ (in.)	0.043	0.043	0.043	0.043	0.043

APPENDIX B:

WEDGE CYLINDER FAILURE LOADS

Table B.1 Wedge cylinder failure loads for zero freeze/thaw cylinders.

Material Type	Failure Load (lbs.)	Repair Material Age (days)	Type of Cylinder
A	31,220	38	Full
	30,620	38	Full
	31,410	38	Full
	30,930	38	Shear
	24,110	38	Shear
	4,210	38	Shear
B	90,790	31	Full
	90,870	31	Full
	81,610	31	Full
	33,860	31	Shear
	40,670	31	Shear
	28,890	31	Shear
C	41,550	23	Full
	42,600	23	Full
	42,940	23	Full
	38,700	23	Shear
	34,060	23	Shear
	36,150	23	Shear
D	44,420	15	Full
	46,590	15	Full
	43,830	15	Full
	30,740	15	Shear
	34,090	15	Shear
	38,860	15	Shear
E	55,940	14	Full
	52,840	14	Full
	52,940	14	Full
	45,020	14	Shear
	43,240	14	Shear
	33,860	14	Shear

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