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FINAL REPORT

Steel Bridge System with Delayed Composite Action

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

Abstract

Development of a bridge system where composite action is developed after the concrete has hardened would reduce the extent of cracking observed in bridge decks while elimination of shear studs would reduce the potential tripping hazard to workers. The objective of this research was to recommend a system which met one or both of these goals to the Nebraska Department of Roads Bridge Division for further evaluation. To this end, a number of component level tests along with two prototype beam tests were performed. Details of the testing are described in the report. Of the alternatives considered, two systems appear to offer the most promise for eventual implementation. The first system utilizes a plastic boot placed over the stud prior to casting of the concrete creating a void around the stud. After the concrete has cured the boot can be punctured or removed and the void filled with an epoxy grout. Upon curing the system will perform as though the studs had been directly cast into the concrete. The second recommended alternative utilizes a single plate welded along the length of the girder protruding vertically in the middle of the flange. This allows workers to place a foot on either side while walking. Rebar is then passed through the plate to be embedded in the concrete deck. A prototype beam utilizing this alternative was tested and performed as though there were complete interaction between the steel and concrete.

Executive Summary

It is known that shear studs can contribute to the formation of cracks in a bridge deck due to the restraint imposed during curing. The cracks generated can allow moisture and road salts to penetrate the concrete and lead to corrosion and deterioration of the deck. In addition, the shear studs pose a serious tripping hazard to workers who must walk on the girders, especially prior to the placement of formwork.

Development of a bridge system where composite action is developed after the concrete has hardened would reduce the extent of cracking observed in bridge decks while elimination of shear studs would reduce the potential tripping hazard to workers. The objective of the research was to recommend an alternative system to the Nebraska Department of Roads Bridge Division for further evaluation.

To this end, research was conducted at the University of Nebraska - Lincoln as a pilot study to identify potential alternatives which would address these safety and cracking problems. This research included a number of component level tests along with two prototype beam tests. The details of the testing are described in Chapter 3. Of the alternatives considered, two systems appear to offer the most promise for eventual implementation.

The first system is referred to as the epoxy grouted stud alternative. This system utilizes a plastic boot placed over the stud prior to casting of the concrete creating a void around the stud. After the concrete has cured the boot can be punctured or removed and the void filled with an epoxy grout. Once the grout cures the system will perform as though the studs had been directly cast into the concrete. Although the epoxy grouted stud alternative was not investigated experimentally, its method of shear transfer is identical to that of conventional construction which would suggest that the behavior would be similar to the conventional construction.

The second recommended alternative is the drop bar system. The primary advantage of the drop bar system is the elimination of the tripping hazard posed by shear studs. The system utilizes a single plate welded along the length of the girder protruding vertically in the middle of the flange. This allows workers to place a foot on either side while walking along the girder to erect the formwork. Rebar is then passed through the plate and embedded in concrete during casting to connect the deck to the steel girder.

A prototype beam utilizing the drop bar alternative was tested and the results were very positive. The behavior of the test specimen was nearly identical to the behavior assuming complete interaction between the steel and concrete.

In the course of the research a number of alternatives were envisioned. While many alternatives were quickly dismissed a number of alternatives were given additional consideration and rejected upon further review. A summary of the rejected alternatives can be found at the end of the report.

Chapter

Introduction 1

1.1 Problem Statement

Cracking of concrete decks is a costly maintenance item. The transverse cracks that form before opening the bridge to traffic allow moisture to penetrate the deck and are responsible, in part, for corrosion and deterioration of the deck concrete. It is well accepted that one of the primary reasons for development of transverse cracks in bridges is the restraint that is provided by such elements as shear studs. During casting and hardening of concrete, the steel section alone resists the forces induced by the dead weight of the slab and composite action is not a consideration. It is after hardening of the concrete that the benefits of composite action are realized. The shrinkage cracking problem occurs during the curing of the concrete. In this period, concrete needs to shrink; however, the restraint provided by shear studs limits the free shrinkage of the concrete. As a result, tensile forces develop in the concrete that give rise to the observed transverse cracking in the deck.

Developing a bridge system where composite action is developed after the concrete is hardened will reduce greatly the extent of transverse cracking observed in bridge decks.

There is another very important reason for developing a system that relies on something other than shear studs to develop composite action. Before placing the formwork, workers often have to walk over the top flange. In the presence of shear studs, there is a likelihood of workers tripping and possibly falling off. This safety issue has resulted in some states requiring that shear studs be welded after placing the formwork. Labor unions are starting to require NDOR to do the same, i.e., weld the shear studs in the field after placing the formwork. Since field welding could result in lower quality the NDOR Bridge Division has recently begun to look at alternatives where composite action could be developed without the use of shear studs.

1.2 Objectives

The ultimate objective of this initiative is to develop a system where composite action is developed using devices other than shear studs. The specific objective of this research project is to identify a system that could provide composite action after the concrete has hardened which can be integrated with the current construction practices and is economically feasible. This project is a pilot study at the conclusion of which one system will be recommended for further evaluation by the Bridge Division.

1.3 Alternative Summary

A number of alternatives have been identified and investigated to asses their potential for success. This section will introduce each of the concepts along with a brief synopsis of the results from the study. Detailed explorations of the alternatives are included in the body of the report.

1.3.1 Epoxy Injection

The first option explored uses epoxy to literally glue the concrete deck to the supporting girders. At the time of deck casting, a hollow tube is embedded vertically in the concrete. This tube provides a conduit for the epoxy which is pumped under pressure after the concrete deck has cured and shrunk. A nipple similar to a grease zerk affixed to the end of the tube facilitates the pressurized pumping.

The pressure will break any adhesive bond between the steel and the concrete, allowing the epoxy to flow between the two, creating a layer of epoxy; thereby gluing the deck to the steel girder.

The main advantage of the system is that it requires minimal modifications to the existing construction methods. It is also an inexpensive alternative. The pumping operation can be carried out efficiently by a single individual using a lightweight pump. Some method would need to be devised to hold the tubes in place during concrete placement.

Based on component testing performed, however, this alternative does not appear to be a viable solution due to the low shear strength of the resulting connection.

For a detailed explanation of this alternative and results of component testing see Section 3.1.

1.3.2 Mechanical Alternatives

A number of mechanical alternatives have been identified. Each of the variations is based on a similar idea. A device is embedded in the concrete but not attached to the steel girder. After the concrete has cured and the desired shrinking has been allowed to take place, the device is connected to the steel girder. Details utilizing welds, bolts, or threaded studs were considered.

An advantage of mechanical alternatives is that devices can be developed which are capable of transferring any amount of force. By varying the size and spacing, mechanical alternatives would be every bit as flexible as shear studs. An additional advantage of a number of the alternatives is that the devices would not be required to be installed until after forming is in place, thereby eliminating the tripping hazard which shear studs pose.

One of the largest disadvantages of this system is cost. Although manufacture of the individual devices would surely be automated, each individual device would need to be attached to the girders after the deck has cured. This would necessarily be a labor intensive process. In addition, these connections would require a great deal of inspection both at the time of construction to verify their quality and as an item of routine maintenance.

A further disadvantage would be the connections themselves. Any weld can provide an incipiency to cracking or corrosion. Currently, shear stud welds are sealed within the deck protecting them. However, placing exposed field welds at the interval required for shear transfer along the entire length of the bridge may well be a recipe for disaster.

1.3.3 Pre-cast Option

A second epoxy alternative used a much more viscous epoxy than that used for the injection alternative. As a result, the epoxy would need to be applied directly to the steel girder itself. This could be done if the deck were composed of pre-cast sections.

This option is more difficult to compare with the others since it requires that a completely different construction method be employed. While there are times when a segmental pre-cast deck can be economically utilized (Price 2000), this economy is not universally assured.

The advantage of using the second epoxy is that it had very high shear strength and performed better at joining the steel to the concrete than the first epoxy.

The epoxy showed such high strength that a prototype beam test was performed. However, this beam test revealed a weakness of all epoxies. There are two components to shear force transfer, vertical and horizontal. Shear studs resist the vertical component of the shear force using a large head. Epoxy, however, must rely on its relatively low tensile strength to resist this vertical component.

1.3.4 COMBINATION

Several alternatives were developed which attempt to combine the advantages of the epoxy and mechanical systems.

1.3.4.1 STUD STRIP

The stud strip alternative is an attempt to modify and improve on the epoxy injection alternative.

Shear studs are set in a strip of fairly flexible material and laid on the top of the steel girder. When the deck is cast, the studs are embedded in the concrete. After the deck has cured, epoxy is pumped between the strip and the steel girder.

The material for the strip can be chosen such that very good adherence to the epoxy is obtained. One obvious choice would be fiber reinforced plastic. However, a number of alternatives could be explored.

A small spacer can be placed between the strip and the girders. This would allow for effective pumping of the thicker epoxy which was found to adhere well to the steel.

Intermittent connectors, with a much greater spacing than current shear studs, affixed directly to the steel girder would be required to resist the vertical component of shear.

An advantage of the system is the elimination of most shear studs and accompanying tripping potential until after the formwork has been placed. The system is modular; each strip would be around six to ten feet long. This reduces cost by allowing several common configurations to be mass produced.

The main disadvantage of this system is the fact that it would require the most research and development. Not only would the epoxy connection need to be investigated, but the entire design and implementation of the stud strip itself would need to be determined.

1.3.4.2 Epoxy Embedded Studs

Under this alternative, the steel girder has conventional shear studs. A formed plastic boot is then placed over the studs, or row of studs. The boot prevents the concrete from embedding the stud and becoming composite. After the deck has cured, the boot is pierced and the void is filled with either epoxy or grout to create the composite action.

The main advantage is due to the fact that very little modification needs to be made to the current construction methods.

The disadvantage would be additional labor required for setting and grouting the boots.

1.3.4.3 Mixed aggregate methods

Several alternatives were examined which used a mix of aggregates to improve the performance. The first of these simply used a layer of sand covering the top flange prior to placing the concrete deck. This method is an extension of the pumped epoxy alternative. The idea is that the concrete will infuse the layer of sand and create an irregular surface. After the concrete has cured and shrunk, epoxy is injected into the layer of sand.

The layer of sand allows the epoxy to flow freely. Coping applied to the edges of the girder embed into the concrete preventing leakage and guide the epoxy along the length of the girder. The irregular interface created by the concrete exposes more surface area to the epoxy, creating a stronger bond. Additionally, the layer of sand prevents the concrete from bonding to the steel surface. It is believed, based on the component testing, that one reason for the extremely low adhesion of the epoxy to the steel after pumping is the cement residue left on the steel after de-bonding has occurred.

A second alternative utilizing aggregate is to glue on a layer of large pebbles in the shop prior to shipment. From this point, the alternative would proceed similar to the previous utilizing a layer of sand followed by pumped epoxy after the deck had cured and shrunk. The irregular surface presented by the pebbles would increase the surface area for the epoxy to adhere to and also provide a degree of mechanical interlock.

This method would allow the use of an epoxy ideal for adhesion to steel surfaces for the pebbles while using the low viscosity epoxy for the pumping. Since the pebbles would be applied in a controlled environment, the connection would be of high quality.

1.3.5 Non-Delayed

In 2001, the Occupational Safety and Health Administration, OSHA, required that all girders be free of shear studs prior to placement of formwork. For this reason, NDOR requested that additional research emphasis be placed on their elimination. Therefore, a system was investigated within this project which did not necessarily ensure delayed composite action, as is the stated purpose of the project. However, the system appeared as a promising alternative to shear studs.

Figure 1-1 illustrates the drop bar alternative. The top layer of transverse, and all longitudinal reinforcement, has been removed for clarity. The primary component of this system is a vertical perforated plate. Semi-circular holes near the top allow transverse reinforcement to be easily placed. Intermittent full through holes allow short length of specially bent transverse bars to drop down and pass through. These drop bars lock the deck to the girders and provide the vertical component of the shear transfer while the regular transverse bars will push against the sides of the semi-circular holes providing the horizontal component of shear transfer.

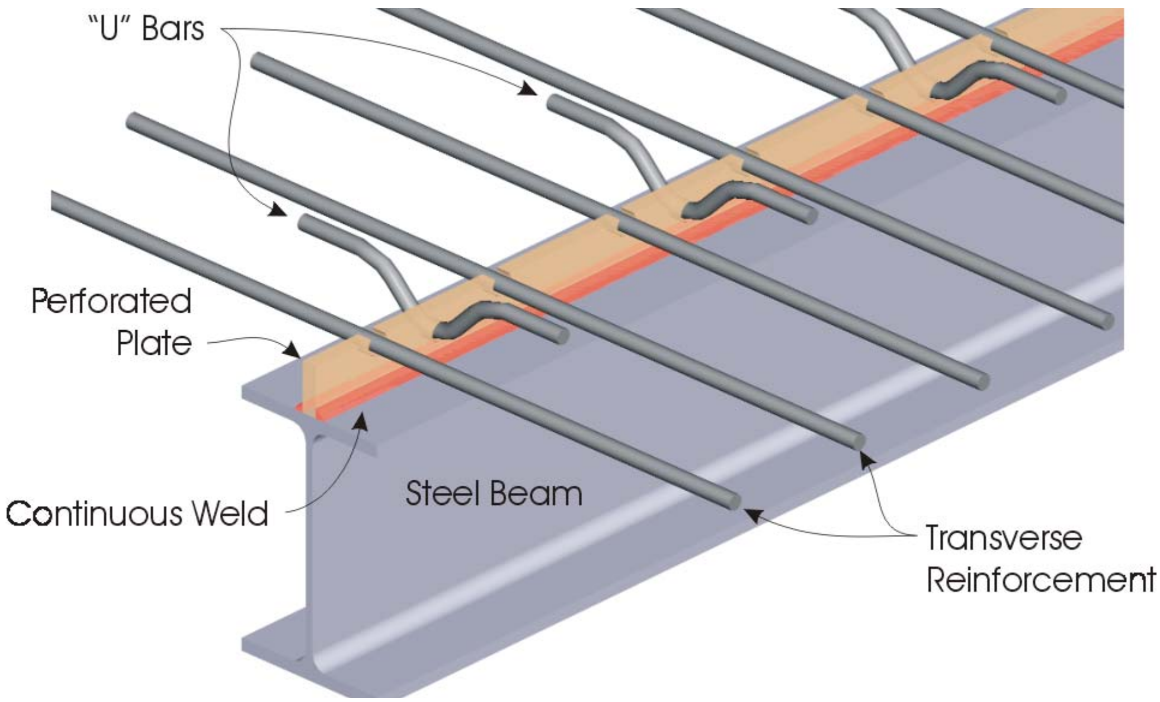

Figure 1-1: Conventional Two-Span Continuous Steel Girder

Although this system does not explicitly delay composite action, the small gaps which necessarily exist between the bars and the edges of the hole may provide the freedom of movement required to prevent restraint cracking of the deck. However, further research is needed to determine the true behavior.

A test was performed on a prototype beam which used this system. The beam displayed good performance, both in strength and ductility.

Several of the advantages of this system are cost and safety. Fabrication of the perforated plate can be highly automated thereby reducing cost. A one sided continuous or intermittent weld can be used to attach the strip to the girder allowing the use of automated welding equipment. The lay out time associated with shear studs is also eliminated.

Although the vertical plate is shop welded on the girder, and therefore can still pose a possible tripping hazard, the flange is clear of obstructions to either side of the plate allowing unobstructed foot placement while walking along the length.

One other disadvantage is the innovative nature of the system. While some similar methods have been investigated (Oguejiofor and Hosain 1992; Medberry and Shahrooz 2002), additional research would be required to prove the performance of the system.

1.4 Testing Narrative

Essentially four phases of testing were performed. Each subsequent phase was in response to the results of the previous. This section presents that progression and outlines the testing which was performed.

The first alternative investigated was epoxy injection. This was chosen due to the minimal modifications required to the existing construction methods. Small scale component testing was performed to determine the potentials of the method. Testing was done to investigate the amount of pressure required to separate the steel from the concrete, how well the epoxy dispersed, the strength of the epoxy and ways of improving that strength. The epoxy appeared to bond well with the concrete but did not perform well when joining the concrete to the steel.

A second type of epoxy, Dexter Hysol 9460, was investigated to determine whether the epoxy concepts should be abandoned all together. Again, small scale component testing was performed to assess the characteristics of the epoxy. The second epoxy appeared to perform much better than the type used for injection. It was specifically recommended by the manufacturer for use when joining two dissimilar materials, such as concrete and steel, and has been used to rehabilitate concrete structures by applying fiber reinforced plastics.

It was decided that a prototype beam test should be performed to determine the viability of the idea that epoxy could be used to transfer shear in a composite beam. The beam was designed to generate a large shear force and exhibit a large amount of inelastic deformation. The slab was pre-cast and the Dexter Hysol 9460 epoxy was used to the glue the concrete slab to the steel girder. Results of the test were disappointing. The slab separated from the steel at a load which was a fraction of the predicted ultimate value. Upon subsequent evaluation of the specimen, it was determined that the vertical component of shear overcame the low tensile strength of the epoxy. Based on this test, it was determined that if epoxy was to be used as a shear transfer mechanism, a secondary mechanism would be required to transfer the vertical component of shear.

Due to the poor performance of the epoxy beam test and the increased urgency in the spring of 2001 to develop an alternative to shop applied shear studs in response to new requirements from the Occupational Safety and Health Administration, a prototype beam utilizing the drop bar alternative was performed. The results of this test were very positive. The behavior of the specimen was nearly identical to the predicted behavior. Complete results of this test can be found in Section 3.7.

The next chapter introduces some of the elementary concepts behind composite action. The third chapter presents more details of the proposed alternatives and the results of the experimental testing which was performed.

Chapter

2 Fundamental Behavior of Composite Beams

Consider a simply supported non-composite beam, as shown in Figure 2-1, composed of a concrete slab lying on top of a steel joist where friction is ignored between the two components. For simplicity, both materials are assumed to behave linear elastically. In reality, concrete cracking due to tension would have to be taken into consideration. When a load is applied at midspan, the two components attempt to bend independently of one another about their own respective neutral axis. Due to this, the bottom fiber of the concrete slab is in tension and the top fiber of the steel joist is in compression. Since the two components are in contact they maintain the same deformed shape, and therefore, curvature. The resulting strain distribution is shown in Figure 2-2a.

Assume now that some mechanism is introduced between the slab and the steel joist which attempts to connect the two components together. This

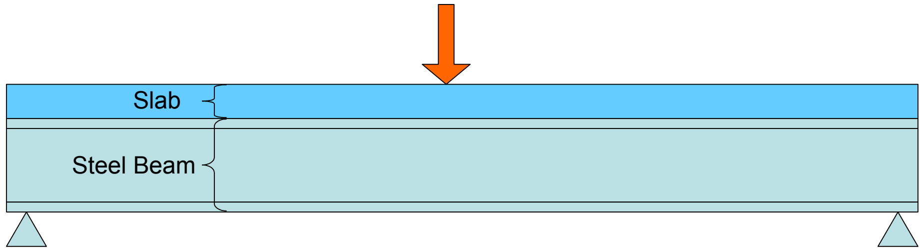

Figure 2-1: Conventional Two-Span Continuous Steel Girder

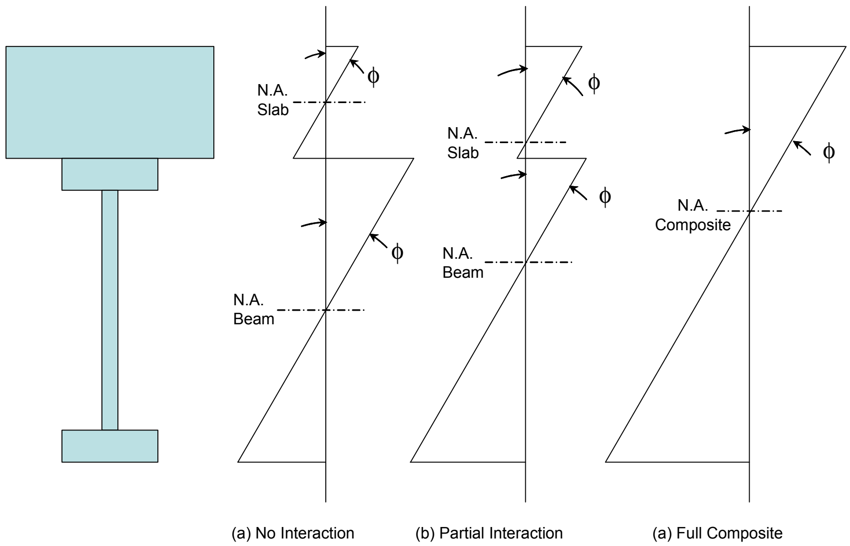

Figure 2-2: Conventional Two-Span Continuous Steel Girder

mechanism produces a compressive force on the bottom fiber of the concrete slab while an equal and opposite tensile reaction is imposed on the top fiber of the steel joist. Again, since the components are in contact the curvature within each component is the same. As seen in Figure 2-2b, these forces, or shear force, cause the neutral axis in the slab to shift downwards while the neutral axis in the joist is shifted upwards.

In the extreme case, the shear transfer mechanism is infinitely rigid and the strain in the bottom fiber of the concrete deck is the same as the strain in the top fiber of the steel joist. This condition is referred to as full composite action and the strain distribution under this condition is depicted in Figure 2-2c. Under this condition, a single neutral axis is located between the neutral axes of the independent components and the strain is linear through the full depth of the section.

A certain amount of ductility is required such that all shear connectors participate in carrying the shear force. Consider the shear diagram of a uniformly loaded beam. The shear at the ends is much higher than the shear in the middle. If the shear connectors were perfectly rigid, a sufficient number of connectors would have to be placed at the ends of the girder to transfer the full shear force without failure. However, if the shear connectors are ductile then as the load is increased and the connectors in the region of high shear deform, the load is transferred inward to those under lower load. In fact, current design based on the flexural strength of the composite section relies on the combined strength of all connectors between the points of maximum and zero moment. The requirements for this design basis is stated by Slutter and Driscoll (1965), "the magnitude of slip will not reduce the ultimate moment provided that (1) the equilibrium condition is satisfied and (2) the magnitude of slip is no greater than the lowest value of slip at which an individual connector might fail."

Chapter

3 Alternative Details

Within this section each of the alternatives are considered in more detail. The results of any experimental testing associated with each alternative are also presented.

3.1 Epoxy Injection

The first option explored using epoxy to literally glue the concrete deck to the supporting girders. Use of pressure injected epoxy to repair concrete structures has been around for many years. However, the idea of using it to develop delayed composite action between a cast-in-place concrete deck and a steel girder to prevent shrinkage cracks is a very new and innovative one.

3.1.1 Detailed explanation of system

At the time of deck casting, a hollow tube is embedded vertically in the concrete. This tube provides a conduit for the epoxy which is pumped under pressure after the concrete deck has shrunk during curing. After casting, the tube is located and a nipple similar to a grease zerk is affixed to the end of the tube to facilitate the pressurized pumping.

After the deck has cured and been allowed to shrink the desired amount, the epoxy is pumped through the tube.

As the epoxy is pumped, the pressure will break any adhesive bond between the steel and the concrete, allowing the epoxy to flow between the two, creating a layer of epoxy thereby gluing the deck to the steel girder.

A series of tubes would be required along the length of the bridge to assure adequate flow.

3.1.2 Testing plan

For the concept to work, strong adhesion between steel and concrete must be provided by injecting epoxy at the steel-concrete interface after the concrete shrinkage has taken place. Many issues have to be addressed in going from concept to implementation and achieving an optimized behavior. Twenty-one tests were conducted to understand and address the following:

1. Concrete-steel bond prior to injection of epoxy
2. Injection process and spreading of epoxy over the interface
3. Shear strength of epoxy
4. Epoxy-steel and epoxy-concrete bond strength

3.1.3 Testing

A two component resin, WEBAC- 4110, with mix ratio A:B = 2:1 was injected or directly applied in all initial tests unless noted. For injection of epoxy, an IP395 electric pump along with type 13-60S, 2" packers manufactured by WEBAC were employed. The pump is capable of producing up to 3000 PSI of pressure.

Test #1 - Measuring Concrete-Steel Bond

During pumping, the epoxy pressure must be enough to break the tension bond but must not be excessive so as to avoid bending induced cracks in the concrete during injection. Therefore, the purpose of this test was to determine the magnitude of bond present between the steel and concrete. Initially two identical specimens were prepared by casting concrete on steel plates. Since the measured tension bond must be representative of a concrete deck in a typical steel girder bridge, the concrete mix was selected to have the same material proportions as those typically used in the concrete deck of bridges. The mix proportions per cubic yard of the mix are:

Cement = 658 lb, Water 331 lb, Coarse Ag. = 875 lb, Fine Agg. = 2041 lb

Figure 3-1 shows the dimensions of the specimens.

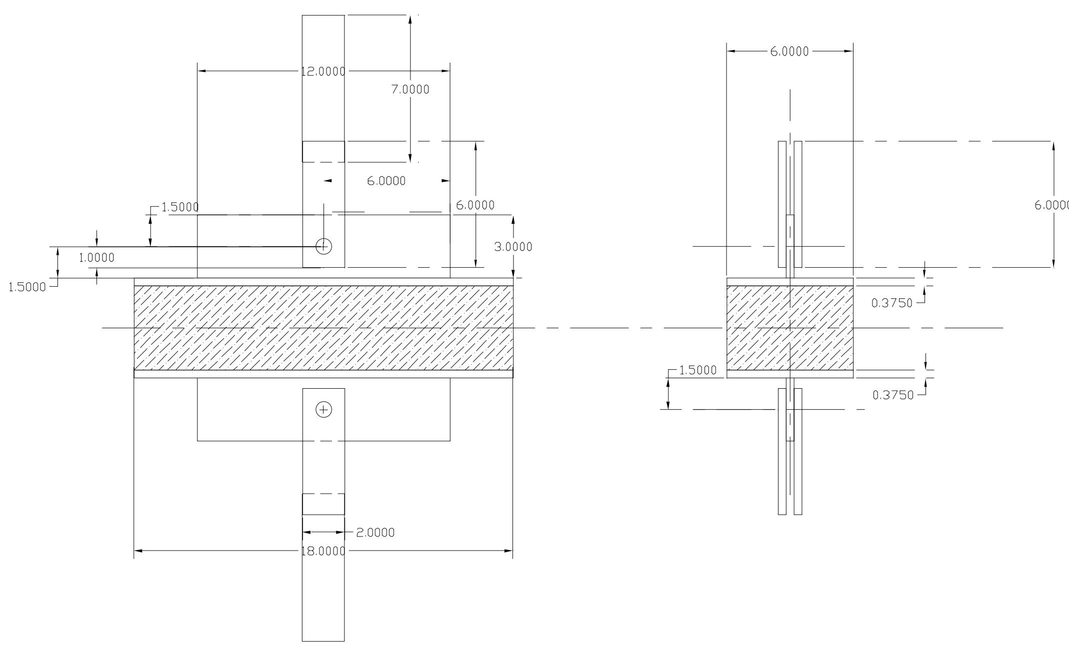

Figure 3-1: Specimen Dimensions (inches)

Testing Procedure:

Figure 3-2 shows a test specimen placed in the universal testing machine. Special handling care was taken to avoid stressing the concrete bond which may cause premature de-bond at the interface. Also, the tension splices were carefully aligned to avoid any eccentricity resulting in bending stresses induced at the interface.

The first specimen was tested at 7 days after the concrete pour and the second specimen was tested after 13 days. Bond strength in the early days is of most importance because injection of epoxy would normally have to take place during this period of time.

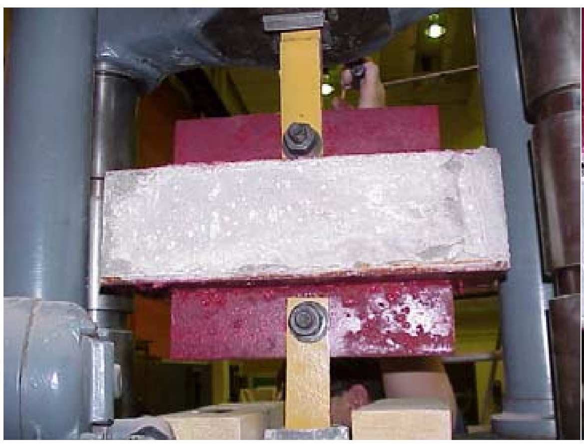

Figure 3-2: Photo of Specimen

Figure 3-3: Specimen after failure

Summary of Test Results:

Figure 3-3 shows a photo of the tension bond failure. The failure occurred only on one of the contact surfaces while the other surface remained intact. The results of the test are listed below.

- 7 days old tension bond failure load measured at 1979 Lbs.
  Contact Area = 96 (in^2)
  Tensile Strength, σ_u= 20 PSI

- 13 days old tension bond failure load measured at 2200 Lbs.
  Contact Area = 6" × 16" = 96 (in^2)
  Tensile Strength, σ_u = 1757/96 = 22.9 PSI

- Compressive strength of concrete using standard cylinder test was measured 3679 PSI.

As noted, the magnitude of the tension bond is only a fraction of the tensile strength of the concrete. Also, the bond strength did not seem to increase with time.

It would later be determined that the bond could be broken locally when epoxy is injected at pressures in magnitude of 500PSI or more, causing progressive fracture of the bond between the cast concrete and the steel.

TEST #2 - MEASURING MAXIMUM STEEL-CONCRETE BOND

Since only one side of each contact surface failed during test #1, an upper bound for the tension bond between steel and cast-in place concrete could be measured by repairing and re-testing the failed specimens. Epoxy was applied to the failed surface of the specimens to connect the steel to the failed concrete surface and allow the failure to take place in the sound contact surface of the specimen. Figure 3-4 shows one of the failed specimens.

Figure 3-4: Specimen after failure

Results:

The tension bond for the two specimens was 35.4 PSI and 38.5 PSI.

TEST #3 - DISPERSION OF EPOXY OVER A RECTANGULAR INTERFACE AREA

This test was aimed at observing the actual dispersion of epoxy at the interface with relation to the width and length of interface area.

Three specimens were prepared by casting concrete on poly-glass sheets with dimensions of 4"×20", 6"×20" and 8"×20". Clear poly-glass was used so that the dispersion pattern could be observed on the interface as epoxy was being injected. One 2" diameter PVC pipe was used for injection of epoxy and access to the interface. Schematics of the 3 specimens are shown in Figure 3-5.

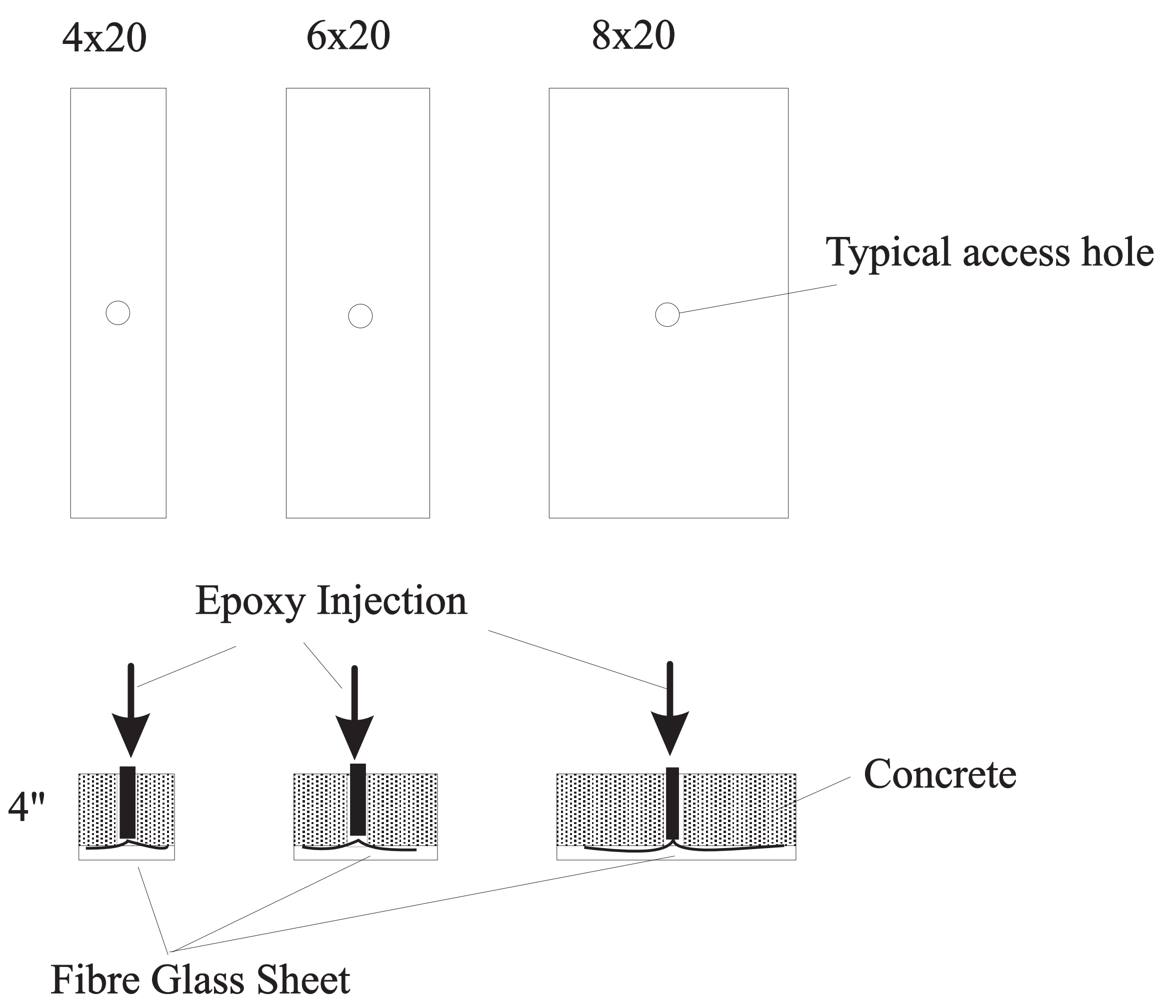

Figure 3-5: Schematic of specimens

Figure 3-6: Underside of all 3 specimens

Test observations and results:

Low pressure epoxy (200 PSI) was injected through the holes and caused separation of the concrete from the poly-glass sheets. Figure 3-6 shows the photo taken from underneath the specimens before injecting epoxy and Figures 3-7 through 3-10 show the progression of epoxy dispersion. As can be noted, dispersion takes place in a perfectly radial fashion. It reaches the long edges of poly-glass interface and leakage starts along these edges before the epoxy can completely spread out toward the short edges.


Figure 3-7: Pumping Epoxy (1)

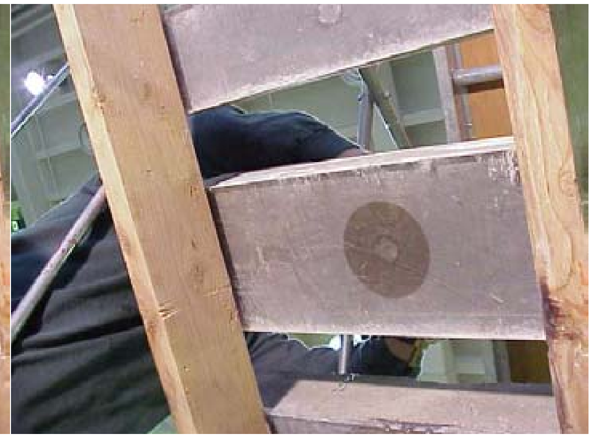
Figure 3-8: Pumping Epoxy (2)


Figure 3-9: Pumping Epoxy (3)

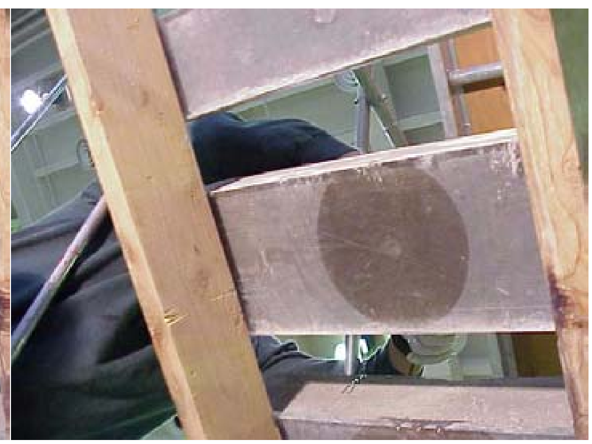
Figure 3-10: Pumping Epoxy (4)

Test #4 - De-bond and Dispersion of Epoxy - Cast-on-Steel

The objective for this test was to simulate the desired de-bond and uniform epoxy spread between the cast-in place concrete deck over the flanges of steel girders. First, the possibility of de-bond of the concrete cast on steel was examined. The second goal was to observe the pattern of epoxy dispersion when significant bond exists between the steel and concrete.

One specimen was prepared by casting 4" thick concrete measuring 16"×16" over a 2" thick steel plate. To allow injection of epoxy, one 2 in diameter PVC pipe was embedded in the concrete.

Test procedure and result

The epoxy pressure was increased to 500 PSI. However, leakage from couplers and packers was observed during the test. At this pressure, de-bond between steel and concrete did not take place. However, some amount of epoxy was able to find its way through the interface. Therefore, the pattern of dispersion could be observed. After two days, the concrete slab was separated from the steel plate. Great care was exercised during the separation process in order to preserve the integrity of the interface. It was observed that the epoxy spread uniformly over a circular area. Figures 3-11 and 3-12 show the specimen after injecting the epoxy and after the specimen had been dismantled, respectively.

Figure 3-11: Specimen after injection

Figure 3-12: Spread of epoxy

TEST #5 - GUIDING EPOXY DISPERSION

A number of previous tests indicated that epoxy dispersion takes place in a radial fashion. To ensure that the epoxy spreads throughout the interface uniformly before leaking at the boundary of the interface, which would cause loss of pressure and prevent uniform spreading, the idea of guiding the epoxy was proposed and examined. A quick examination of Styrofoam pieces subjected to pressure injected epoxy revealed that under moderately high pressure the porosity of Styrofoam would allow passage of the epoxy. Narrow strips of Styrofoam were then used as channels to guide the epoxy in the long direction of the interface. For comparison, 3 specimens identical to those used in Test #3 were prepared, however a narrow strip of Styrofoam was placed at the bottom end of the PVC pipe in each specimen. Different joint configurations were used to see the effectiveness of the epoxy transfer from the pipe to the Styrofoam strip. For the 4"×20" and the 6" ×20" specimens, a pair of notches was introduced at the bottom end of the pipe along the Styrofoam strip, while no modification was made to the pipe in the large specimen. Figure 3-13 shows the arrangement of the Styrofoam strips and the end condition of the pipes before casting the concrete.

Figure 3-13: Setup showing Styrofoam guides

Test Observations and Results

As the bond between the concrete and the poly-glass is weak, moderately low pressure (300 PSI) was used to break the bond and inject epoxy over the interface. Figures 3-14 through 3-16 show the progression of epoxy spreading. It was observed that dispersion takes place in a radial fashion and is not affected by the presence of the Styrofoam strips, or the presence of a notch at the pipe. Also, the flow of epoxy between the Styrofoam and poly-glass progressed at the same rate as in other areas of the interface.

Figure 3-14: Underside of specimen

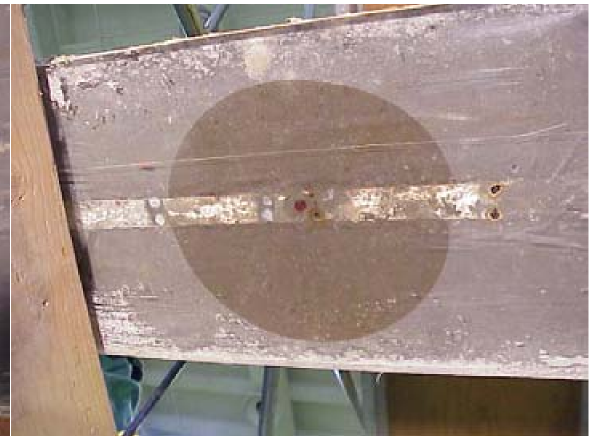

Figure 3-15: Pumping Epoxy (1)

Figure 3-16: Pumping Epoxy (2)

Test #6 - De-bond and Dispersion- Cast- on Steel Plate

As in Test #4, high epoxy pressure could not be maintained due to leakage. Another cast-in-place specimen of the same size was prepared to examine the de-bond between the concrete and steel and find the dispersion pattern. No specific means of channeling epoxy was provided and no notches were made at the bottom end of the pipe. Figure 3-17 shows the specimen before casting concrete.

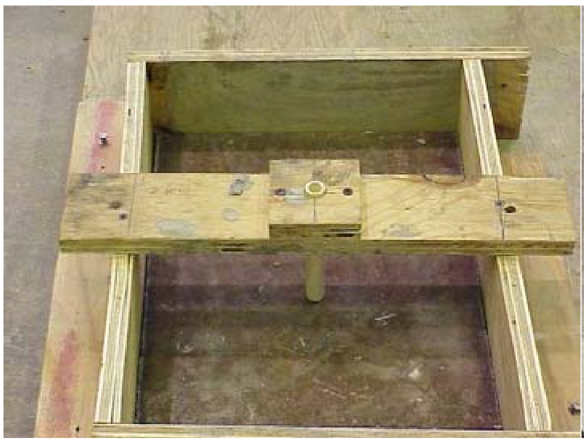

Figure 3-17: Form with injection tube in place

Figure 3-18: Dismantled Specimen

Test Observations and Results

The epoxy under pressure of about 1000 PSI was sufficiently high to cause de-bond and penetrate through the interface. A small amount of injection was permitted to preserve the pattern of dispersion as opposed to allowing complete spreading which would have covered the whole area. After two days, the concrete and steel were manually separated and a circular area of hardened epoxy bonded to the steel was observed, indicating radial dispersion during injection (Figure 3-18).

Test #7 - Sealing Interface Boundary

In the previous tests, the leakage of epoxy at the boundary of the interface was observed. The loss of pressure due to leakage at the boundary points near the pipe occurred before the epoxy reached the far points of the interface. Preventing and delaying the leakage was deemed necessary to enhance uniform spreading and flow of the epoxy throughout the area.

This test was aimed at experimentation and examining the ability of two different types of sealant in preventing leakage at the boundary of the interface. For expedience, a precast 6"×16"×8" concrete block was drilled through at the center. The concrete block was placed on a steel plate and two different types of caulking (MD Structural Adhesive and MD Silicon-Based Caulk) were applied along two edges for comparison. Small openings were left at the corners to allow passage of air pockets. Figures 3-19 and 3-20, respectively, show the specimen and close-up shot of the caulking agents. The Silicon-Based product is white (front edge) and the Structural Adhesive is yellow (side).

Figure 3-19: Test Setup after caulking

Figure 3-20: Close-up of Caulk bead

After 2 days to allow the caulking agent to set, epoxy was injected under low pressure. Injection was stopped when considerable leakage took place, indicating that the entire interface most likely was covered by epoxy. The specimen was taken apart after one day to allow sufficient time for hardening of the epoxy and to observe the extent of the epoxy coated area.

Test Observations and Results

As the concrete was already precast, there was no initial bond between the steel and concrete. Epoxy was pumped under low pressure (500 PSI) and allowed to flow throughout the interface and break the seals. The MD Silicon-Based Caulk performed better than the Structural Adhesive by exhibiting more flexibility and better containment of the epoxy under higher pressure. Figure 3-21 shows the leakage along the boundary.

Figure 3-21: Specimen after injection

After one day, the concrete block was separated from the steel plate. It was observed that the region of interface was completely bonded by the epoxy, indicating that during the injection process, caulking provided sufficient confinement and permitted full spreading of epoxy throughout the interface. Figure 3-22 shows the epoxy bonded surface after the removal of the concrete.

Figure 3-22: Specimen showing spread of epoxy

TEST #8 - GUIDING EPOXY DISPERSION AND SEALING INTERFACE BOUNDARY

This test was conducted in continuation of Test#5, again with the objective of improving epoxy flow throughout the interface before leakage starts at the boundary. However, in this test, in addition to caulking agents, Styrofoam strips were used to guide the epoxy. Three specimens of size 4×20, 6×20 and 8×20 were prepared. Figure 3-23 shows the underside of the specimen prior to pumping the epoxy. BD Silicon-Based Caulking agent was used to seal along the boundary of the interface.

Figure 3-23: Underside of Specimen

Figure 3-24: Pumping Epoxy

Test Observations and Results

Moderately low pressure (300 PSI) was used to break the concrete-polyglass bond and inject the epoxy throughout the interface. It was observed that the epoxy spread in a radial fashion, away from the PVC pipe. Flow of the epoxy through the Styrofoam channel was not observed outside the circular area where the epoxy had flowed in a radial fashion between concrete and polyglass (Figure 3-24). It was also observed that once the epoxy reached the boundary it was blocked by the caulking agent and moved in the longitudinal direction of the polyglass, confirming the effectiveness of the seal. Leakage started only when the epoxy reached the far end of the boundary, and the whole area of the interface was completely covered.

TEST #9 - SEALING INTERFACE BOUNDARY - CAST-IN-PLACE CONCRETE

Two tests were conducted in continuation of Test #7. The objective for testing two cast-in-place specimens on different sizes of steel plate was to examine the effectiveness of sealing the boundary in preventing leakage and allowing uniform distribution of the epoxy over the interface.

Specimen A (16"×16"×4") and Specimen B (6"×16"×8") were both cast on steel plates. After 5 days, BD Silicon-Based caulking agent was applied along all edges, leaving small openings at the corners as vents. Narrow Styrofoam strips were once again used to aid in guiding the epoxy on Specimen B only.

After two days to allow the caulking agent to set, epoxy was injected at over 1500 PSI. De-bond took place in both specimens and flow of epoxy from the corner openings was observed.

Injection was stopped when considerable leakage had taken place, indicating that the entire interface most likely had been covered by epoxy. After one day, the epoxy had cured and the concrete was separated from the steel plates to observe the extent and pattern of the epoxy covered area.

Test Observations and Results

The silicon-based caulking broke under moderately high pressure right after the concrete de-bonded from the steel base. Figure 3-25 shows a specimen prior to pumping the epoxy and Figure 3-26 shows the flow of epoxy after breaking the seal.

Figure 3-25: Test Setup preparing to pump epoxy

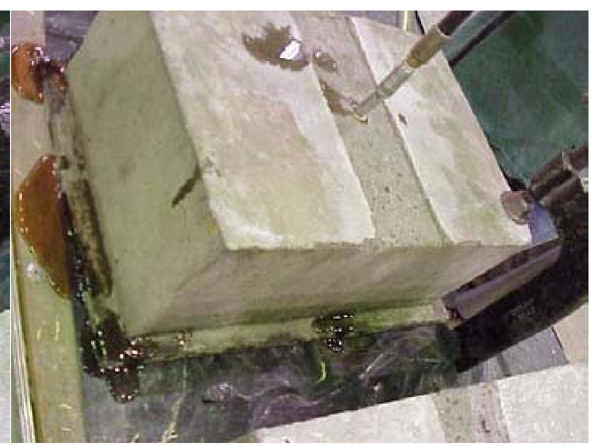

Figure 3-26: Pumping Epoxy

After one day, the concrete block was separated from the steel plate. For both specimen A and B it was observed that the region of the interface was completely bonded by epoxy, indicating that during the injection process the caulking provided sufficient confinement and permitted full spreading of the epoxy throughout the interface. This can be seen in Figures 3-27 and 3-28. Also the embedded Styrofoam strip did not seem to guide the epoxy in the longitudinal direction.

Figure 3-27: Specimen Dismantled showing spread of epoxy

Figure 3-28: Close-up of interface

TEST #10 - SHEAR STRENGTH TEST- PRE-CAST CONCRETE

The objective for this test was to develop an expedient method of measuring the shear strength and identifying potential failure modes for the epoxy bond to both the steel and the concrete. The main advantages of this method of testing are: 1-It does not require time for the concrete to set as in the case of cast-in-place specimens; 2-Epoxy is directly and uniformly applied on the contact area of the steel and concrete, instead of using an injection process, which eliminates any potential problems which may be attributed to the injection process.

Two specimens were prepared and tested. For each specimen, the contact surface of two 4"×4"×16" concrete blocks and both sides of a 2"×6"×16" steel plate were coated with epoxy and all were bonded together. Specimen A was tested after 2 days and Specimen B after 8 days.

A vertical load was applied to the middle steel plate to produce shear stresses over the bonded surfaces. Figure 3-29, shows a photo of the specimen during the test

Test Observations and Results:

- Failure load for Specimen A = 33,119 Lbs
- Failure load for Specimen B = 43,200 Lbs
- Epoxy Bonded area = 2 (15×4) = 120 in^2

- 2-day shear strength = 275 PSI
- 8-day shear strength = 360 PSI

In both specimens, the failure occurred after a layer of epoxy on one side delaminated from the steel, followed by a diagonal tension failure as well as delaminating of the epoxy on the other side of the steel plate.

Figure 3-29: Specimen being tested

Figure 3-30: Pumping Epoxy (1)

Figure 3-31: Pumping Epoxy (2)

On both concrete blocks, the surface layer of epoxy bonded to the concrete appeared intact. Other than the diagonal tension failure in the concrete which had resulted from unbalanced loading, no other plane of failure in the concrete, particularly near the epoxy layer, was observed. Figure 3-30 shows the failed bonded side of the steel plate, and Figure 3-31 shows the condition of the epoxy layer after removing the specimen from the testing machine.

TEST #11 - BENDING TEST

This test was intended to quickly compare the epoxy strength to that of the concrete, and find whether the failure takes place in the concrete, the epoxy, or the interface. It was deemed that the failure mode could indicate where the weakest link would be.

A simply supported beam was constructed by head-to-head epoxy bonding two long pre-cast concrete blocks (4"×4"×16"). The 24" span was supported by rockers at each end. The system was loaded at mid-span, as shown in Figure 3-32, until failure occurred. Figure 3-33 shows the failed specimen.

Figure 3-32: Underside of specimen

Figure 3-33: Photo of specimen failure

Figure 3-34: Close-up of failure

Test results and observations:

Under 452 lbs of load, tension failure occurred. Given the dimensions of the cross section, this value of the load corresponds to 216 PSI tensile stress. As Figure 3-34 shows, the plane of fracture was in the concrete at mid-span and not in the interface. This indicates that the strength of the tensile bond between the epoxy and the concrete as well as the tensile strength of the epoxy itself are both greater than the tensile strength of the concrete.

3.1.4 Steel Only Testing

Since epoxy delaminating from the steel plate was a typical failure mode, to more closely examine the effect of the steel surface condition in delaying de-bonding of the epoxy, a series of tests was conducted. These tests were aimed at:

1. Measuring the maximum shear strength of the epoxy by eliminating, or delaying de-bonding of the epoxy from the steel
2. Experimenting with different surface conditions of the steel to find the optimum condition to increase the bond strength.

Test #12- Bond Shear Strength (Epoxy-to-Steel)

This test was aimed at determining the shear bond strength between the steel and the epoxy when the steel surface was simply clean of mill scale.

Two double shear lap joint specimens with a contact area of 3"×5" were prepared. The steel surfaces in contact were coated with epoxy and tested after 3 days.

Test results and observations:

In both tests, the epoxy delaminated from the steel. The failure loads for the two specimens were: 22,238 Lbs and 21,198 Lbs. For the total shear area = 3"×5"×2 = 30 in^2, the corresponding shear strength was: 741 PSI and 707 PSI.

Test #13- Shear Strength (Wire Mesh)

This test was aimed at determining the shear strength of the epoxy. It was proposed that welding a layer of wire mesh to the steel surface would force the plane of failure to be in the epoxy as opposed to the bond surface.

One of the double shear joint specimens used in Test#12 was modified by welding a wire mesh to all 4 contact areas. The steel surfaces in contact were coated with epoxy and tested after 3 days. Figure 3-35 shows the specimen before testing.

Figure 3-35: Specimen being tested

Test results and observations:

The specimen exhibited a fairly low shear capacity and the failure occurred at 3,388 Lbs.

It was evident that a premature failure of the wire mesh itself was responsible for the failure of the specimen.

TEST #14- BOND SHEAR STRENGTH (GROUND STEEL SURFACE)

This test was aimed at determining the shear strength of the epoxy bond when the condition of the steel surface was enhanced by grinding to create fine grooves in the transverse direction on the steel. To accelerate the hardening of the epoxy, the new type B from WEBAC-1410 was used. Two specimens with a contact area size of 2"×4", per side of plate, were prepared by directly applying epoxy, and tested after 7 days.

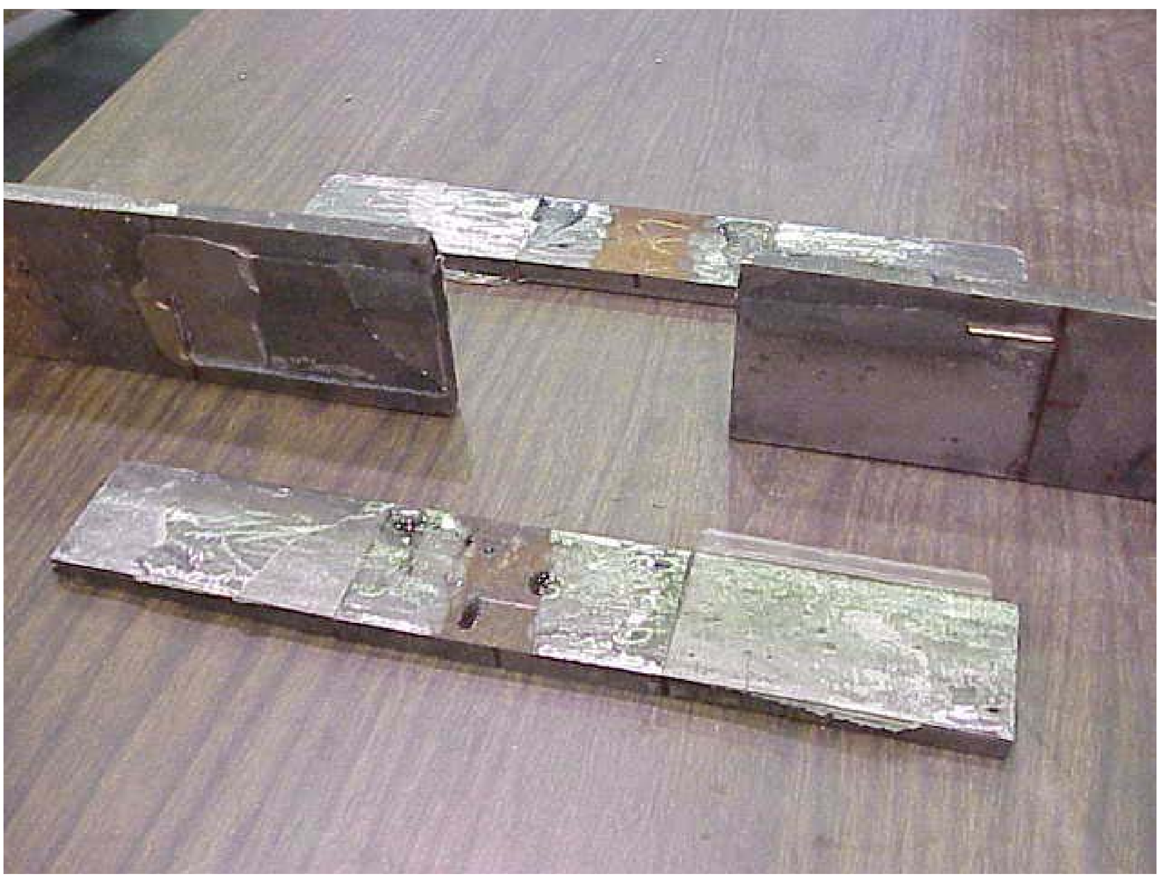

Figure 3-36: Specimen after failure

Test results and observations:

It was observed that the epoxy was de-bonded from the steel and caused the failure of the specimen. This can be seen in Figure 3-36. Only some part of the area of the steel remained in contact. The failure loads of the specimens were 13,180 lbs and 17,000 lbs. This corresponds to shear strengths of 824 PSI and 1,063 PSI.

TEST #15- SHEAR-BOND STRENGTH (MACHINE-GROOVE STEEL SURFACE)

This test was aimed at determining the shear strength of the epoxy bond when significant roughness was created by machine grooving the steel. Transverse grooves of 1/32" depth were made by partial band-sawing all surfaces of the steel in the contact area. Figure 3-37 shows a photo of the specimen. The specimen was tested after 7 days.

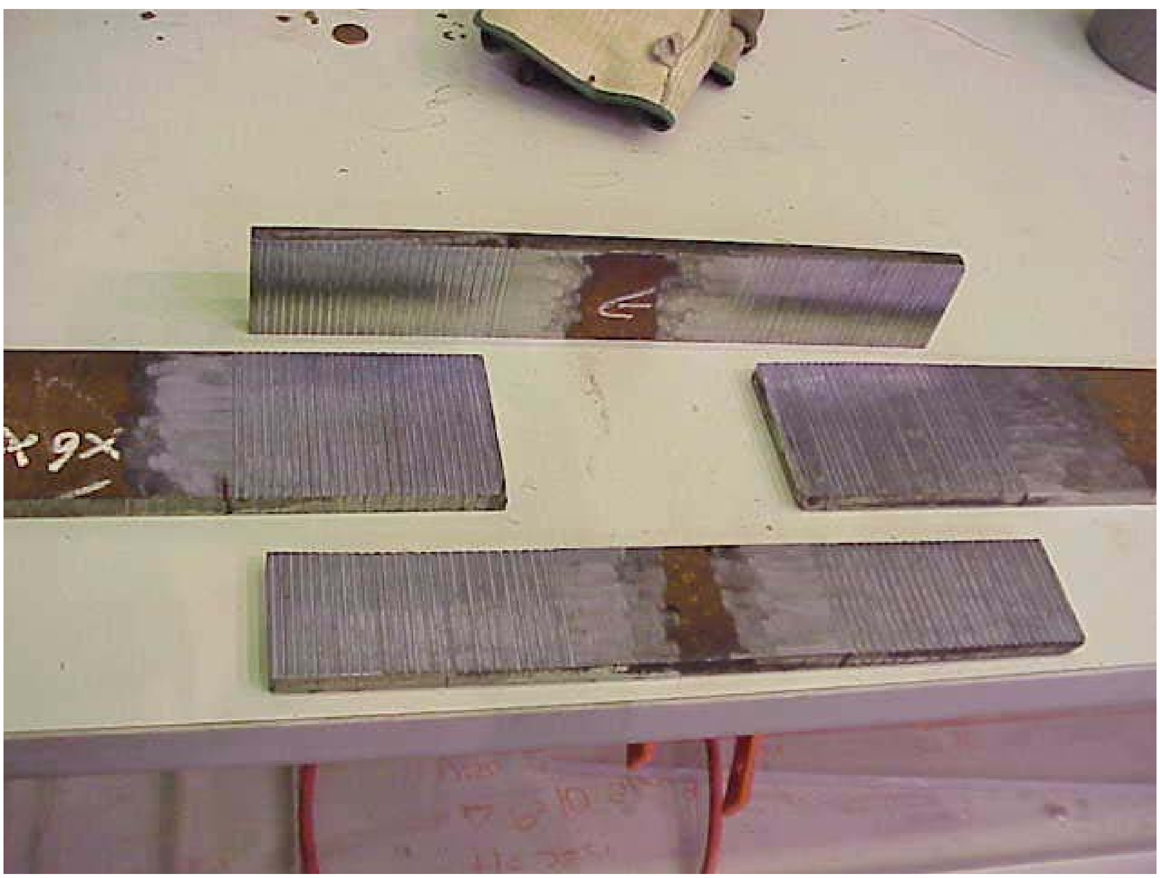

Figure 3-37: Specimen displaying grooved surface

Test results and observations:

The epoxy failure occurred as result of partial de-bonding of steel from steel and partial fracture within the epoxy. The failure load was measured 15,792 lbs, corresponding to 987 PSI. Figure 3-38 shows the failure surface.

Figure 3-38: Close-up of failure

TEST #16- BOND SHEAR STRENGTH (MACHINE-GROOVE STEEL SURFACE)

Examining the partially fractured surface of the epoxy in the previous test revealed that:

1. The steel surface was not completely covered with epoxy. Epoxy could flow into the deep grooves and spread uniformly on the surface.
2. The steel surface was not cleaned using a de-greasing substance before applying the epoxy.

This test was aimed at measuring the shear bond strength of the epoxy on a machine-grooved surface. 1/32" deep grooves were used on all contact surfaces. Also, a de-greaser was used to clean the machine-groove surfaces before applying the epoxy. The specimen was tested after 7 days. Figure 3-39 shows the photo of the specimen during the test.

Figure 3-39: Close-up of lap specimen

Test observations and results:

The specimen failed under a load of 23, 635 lbs, corresponding to 1,477 PSI for the total 16 in^2 area of epoxy. It was observed that the plane of fracture was mostly within the epoxy. Figure 3-40 shows the photo of the failed epoxy surface.

Figure 3-40: Close-up of failed surface

3.1.5 Epoxy Types

The following tests were conducted using different types of epoxy.

Test #17- Bond Shear Strength of Sikadur Gel (Ground Steel Surface)

This test was aimed at examining the shear bond strength of Sikadur Injection Gel. The gel was prepared by mixing the two components, A and B, with mix ratio A:B=1:1. The same size of the specimen as those used in the previous tests with a 2"×4" dimension of contact area was also used here. The steel surface was ground before applying the gel. The specimen was tested after 5 days.

Test observations and results:

The specimen failure load was measured at 6,344 lbs, corresponding to a shear strength of 397 PSI. The failure was sudden, and resulted from debonding of the gel adhesive from the steel. Figure 3-41 shows the failed specimen.

Figure 3-41: Close-up of failure

TEST #18- BOND SHEAR STRENGTH OF SIKADUR GEL (MACHINE-GROOVE STEEL SURFACE)

This test was aimed at finding the shear bond strength of the Sikadur Gel applied over a machine-grooved surface. The same mix ratio and specimen size was used as in Test #17. The steel surface was machined to make 1/

32" deep grooves at 1/8" spacing. The specimen was tested after 4 days. Figure 3-42 shows the picture of the specimen during the test.

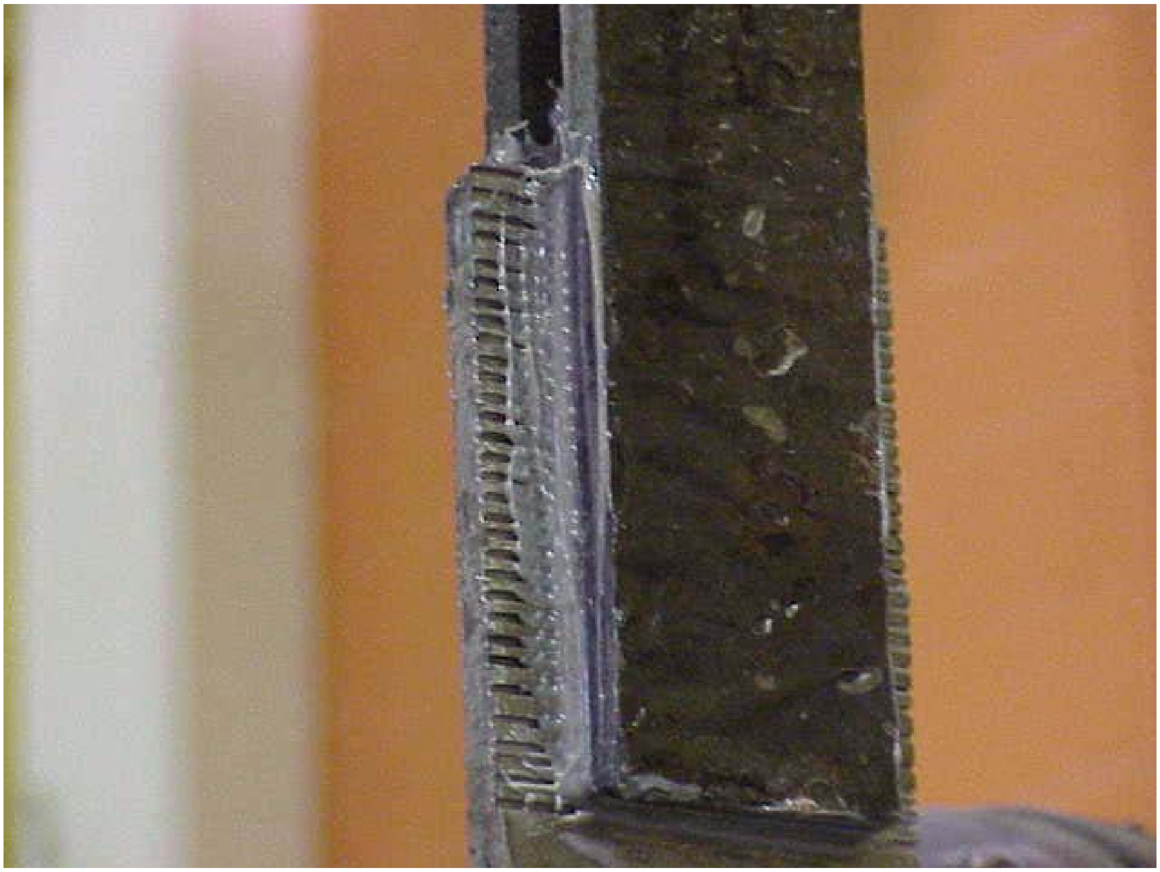

Figure 3-42: Close-up of specimen

Test observations and results:

The specimen failure load was 8,433 lbs, corresponding to a shear strength of 527 PSI. The failure was rather premature and unexpected. Examination of the gel adhesive revealed that part of the area was not completely cured. The failure surface is shown in Figure 3-43.

Figure 3-43: Specimen after failure

TEST #19- BOND SHEAR STRENGTH OF PRIME GEL 2000 (GROUND STEEL SURFACE)

This test was aimed at examining the shear bond strength of Prime Resins - Prime Gel 2000. The gel was prepared by mixing the two components, A and B, with a mix ratio A:B =2:1 by volume. The same size specimen as those used in previous tests, with a 2"×4" dimension of contact area, was also used here. The steel surface was ground before applying the gel. The specimen was tested after 3 days.

Test observations and results:

The specimen failure load was measured at 10,420 lbs, corresponding to a shear strength of 651 PSI. The failure was sudden and resulted from debonding of the hardened gel from the steel. Figure 3-44 shows the failed specimen.

Figure 3-44: Specimen after failure

TEST #20- BOND SHEAR STRENGTH OF PRIME GEL 2000 (MACHINE-GROOVE STEEL SURFACE)

This test was to determine the shear bond strength of Prime Resins - Prime Gel 2000 applied over a machine-grooved surface. The same mix ratio and specimen dimensions were used as in Test #19. The steel surface was machined to make 1/32" deep grooves with 1/8" spacing. The specimen was tested after 5 days. Figure 3-45 shows the picture of the specimen during the test.

Figure 3-45: Test setup

Test observations and results:

The specimen failure load was measured at 17,158 lbs, corresponding to a shear strength of 1,072 PSI. The shear plane of fracture was partly in the hard gel. Part of the plane of fracture was at the steel surface, indicating that gel debonding from the steel was also responsible for the failure. Figure 3-46 shows a close-up of the failure surface.

Figure 3-46: Close-up of failure

3.1.6 Push-Out Testing

In tests #3 through #9, the injection process and different measures to ensure uniform distribution of the epoxy over the interface were investigated. Also, the bond shear strength of the WEBAC 4110 epoxy resin was determined in Tests #10 through #16. The objective of this test was to use the knowledge and skills acquired from the previous tests and apply them to a short beam to develop composite action, and measure the failure load and shear strength of the epoxy bond using a double shear configuration. For this purpose two test specimens were built and tested.

Test specimens:

Four pieces of 24" long W8×13 beams were bead blasted to remove the mill scale. A 6" thick concrete deck was cast on one side of the beam. One ½" PVC pipe per specimen was embedded in the concrete deck for epoxy injection. After 3 days, the forms were removed and BD Silicon-based caulk was applied along the flanges. Seven days after casting of the concrete, epoxy was injected through the access holes. At 1500 PSI, de-bonding of the concrete from the steel flange and leakage of the injected epoxy along the flange was observed, indicating the epoxy had completely spread over the interface.

After 40 days, each beam was flame cut along a line 2" off of the web centerline. Therefore, each I beam was turned into a T-section with a cast-in place concrete deck attached to its flange. Two specimens were assembled by overlapping the webs of two beams and bolting them together, forming a new I beam having both flanges covered with a concrete slab.

Test Set-up

To conduct a double shear test, a point load is applied at the center of the specimen over the web. The reaction produces a uniform pressure over the concrete block. The resultant of the reaction force is at the centroid of the concrete slab section. The eccentricity of the resultant causes bending stresses over the interface which is supposed to be only subjected to pure shear. To prevent the bending stresses, a yoke device is used. The device consists of a pair of channels on each side of the specimen and two Dywidag bars. The horizontal clamping force developed in the device produces a bending moment over the interface counteracting the effect of the bending caused by the vertical load.

Figures 3-47 and 3-48, respectively, show the schematics and photo of the specimen and the yoke device.

Dial gages were installed at both top and bottom ends of the specimens to measure relative movement of concrete with respect to steel.

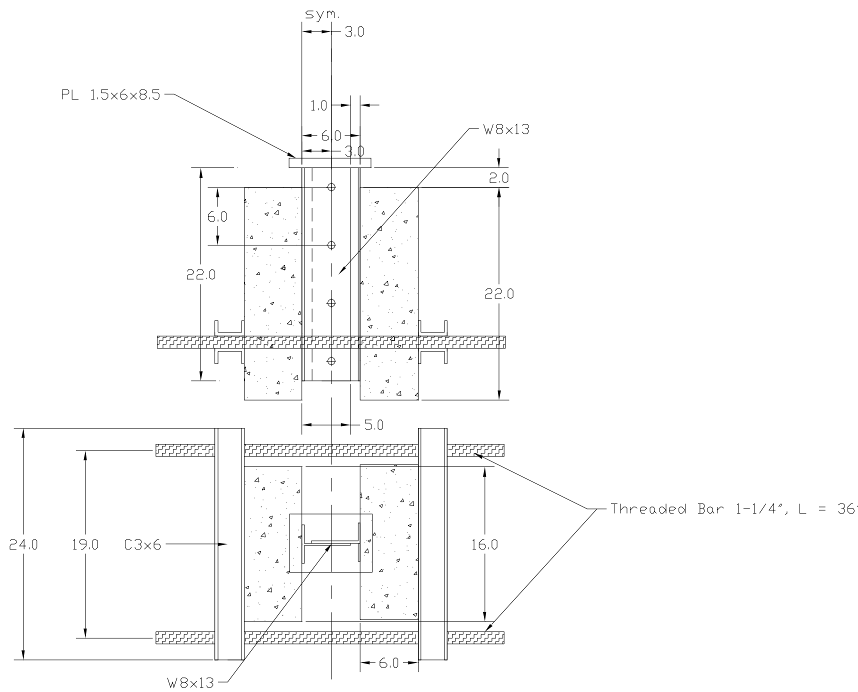

Figure 3-47: Specimen dimensions

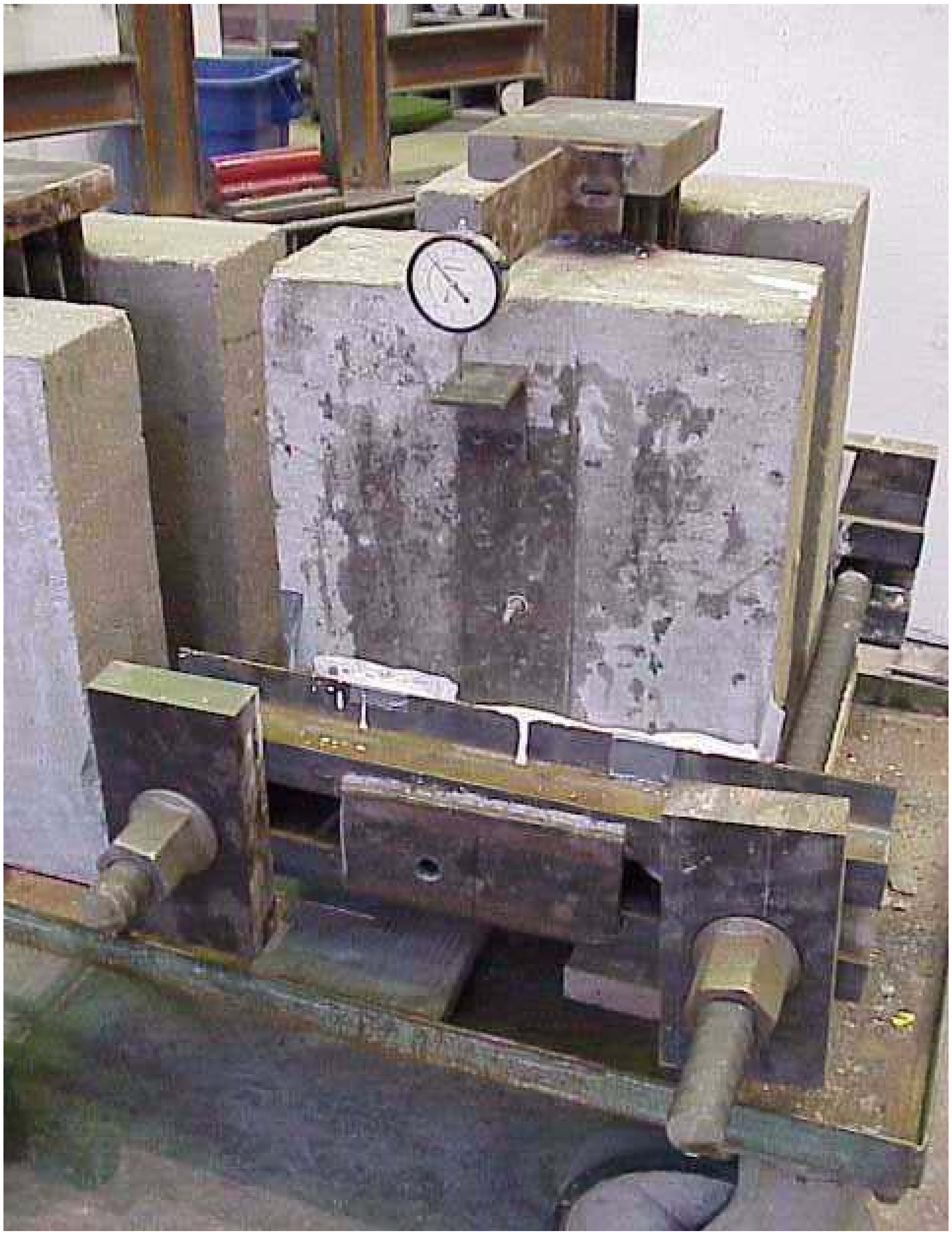

Figure 3-48: Test specimen

Test observations and results

At 40 days past the date of epoxy injection, both specimens were tested. At one end of each concrete slab a single crack was observed on the end face before conducting the test. These cracks were along a transverse line parallel to and very close to the beam flange. It is possible that these cracks resulted partly from concrete shrinkage that took place after the epoxy was injected.

The first specimen exhibited a high initial stiffness and the load increased up to 35,000 lbs. This was followed by a substantial load reduction and increased displacement, indicating distress and failure of the specimen. The second specimen exhibited very similar behavior and failed at 30,000 lbs.

Figure 3-49: Specimen after failure

After moving the cross-head of the testing machine and examining the specimen, it was observed that the in-plane shear failure took place mainly in the concrete and partly at the interface, parallel to the flanges of the steel section. The failure surface can be seen in Figure 3-49.

Figures 3-50 and 3-51 show the force-displacement plots for the tested specimens. Given that the area of shear interface = 2 (20"×4") = 160 in^2, the shear strength of Specimen #1 = 218.8 PSI and Specimen #2 = 187.5 PSI.

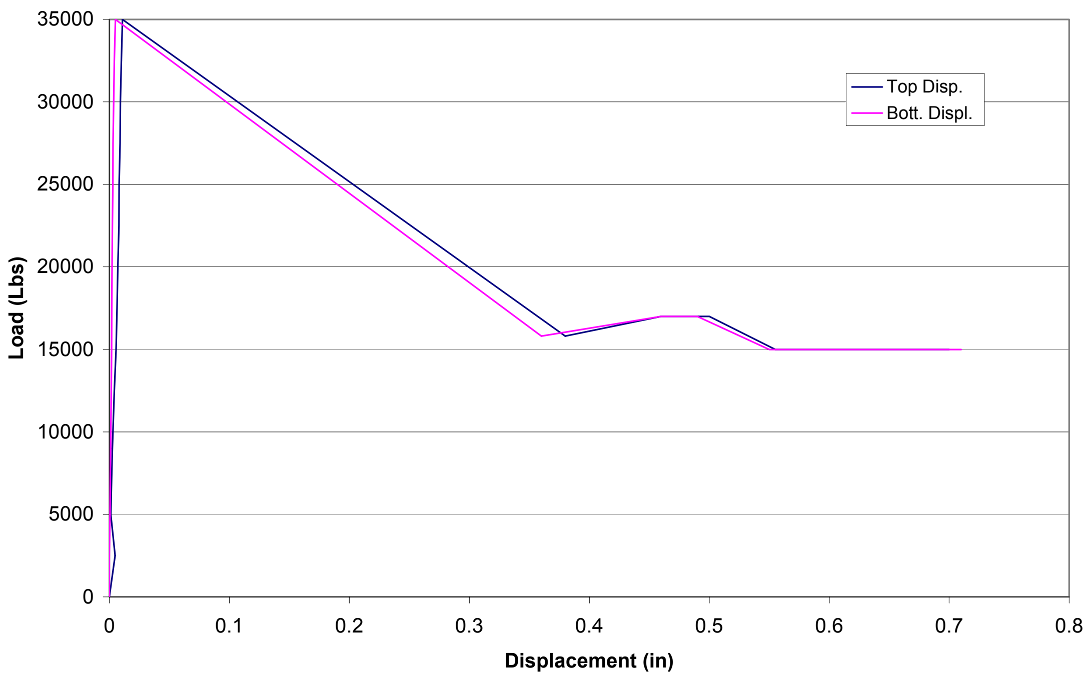

Figure 3-50: Load Deflection Push Out Specimen 1

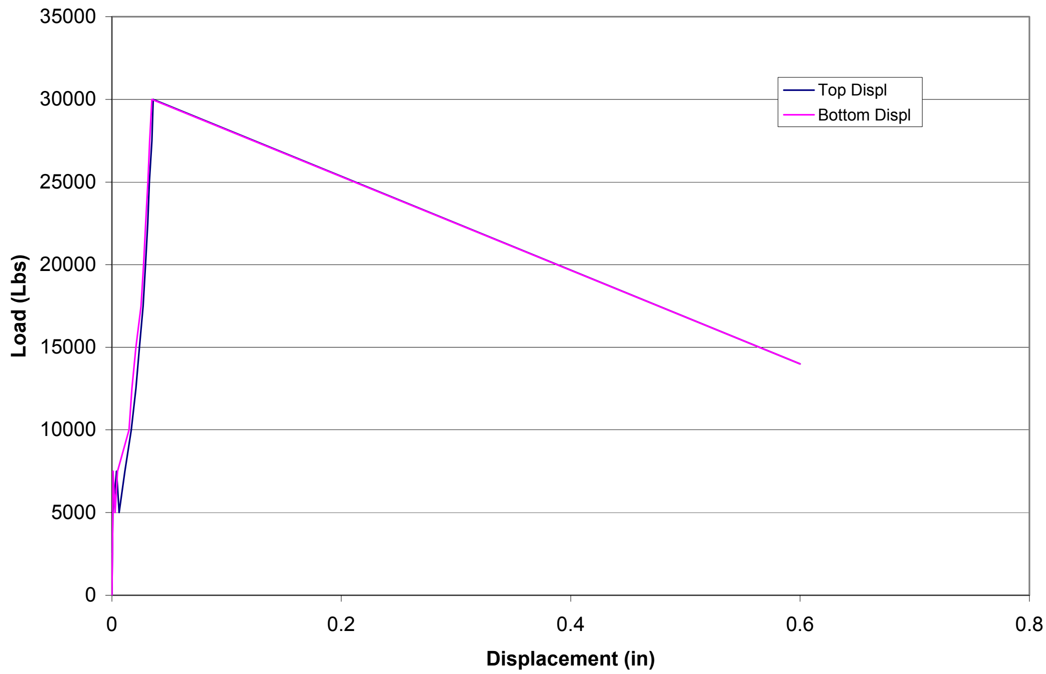

Figure 3-51: Load Deflection Push Out Specimen 2

3.1.7 Summary Results

Based on the results of the experimental testing, it would appear as though epoxy injection is not a viable alternative. Although many of the difficulties encountered with respect to the injection operation were overcome, the strength of the resulting connection is insufficient to transfer the required forces.

3.2 Mechanical Alternatives

A number of mechanical alternatives have been identified. Figure 3-52 illustrates the basic concept. Each of the variations is based on a similar idea. A device is embedded in the concrete, however not attached to the steel girder. After the concrete has cured and the desired shrinking has been allowed to take place, the device is then connected to the steel girder.

The detail shown in Figure 3-52 utilizes a weld to complete the connection; however details utilizing bolts or threaded studs were also considered.

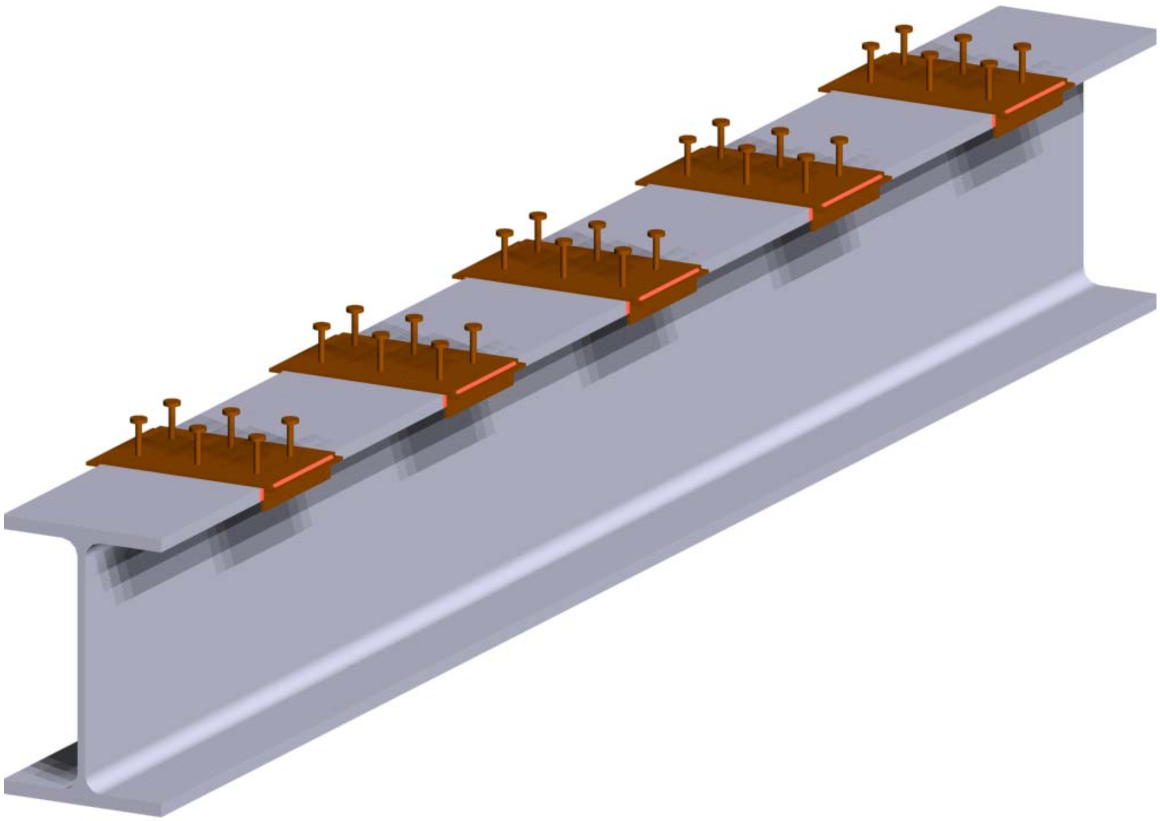

Figure 3-52: Typical Mechanical Alternative

An advantage of mechanical alternatives is that devices can be developed which are capable of transferring any amount of force. By varying the size and spacing, mechanical alternatives would be every bit as flexible as shear studs. An additional advantage of a number of the alternatives is that the devices would not be required to be installed until after the forming is in place, thereby eliminating the tripping hazard which shear studs pose.

One of the largest disadvantages of this system is cost. Although manufacture of the individual devices would surely be automated each individual device would need to be attached to the girders after the deck has cured. This would necessarily be a labor intensive process. In addition, these connections would require a great deal of inspection both at the time of construction to verify their quality and as an item of routine maintenance.

A further disadvantage would be the connections themselves. Figures 3-53 and 3-54 are close up renderings of the connection details. Any weld can provide an incipiency to cracking or corrosion. Currently, shear stud welds are sealed within the deck protecting them. However, placing exposed field welds at the interval required for shear transfer along the entire length of the bridge is undesirable.

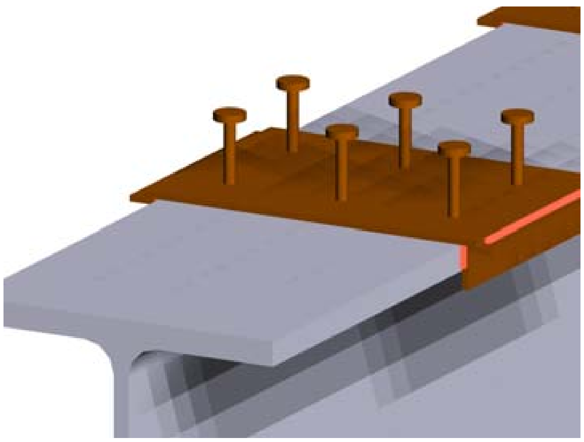

Figure 3-53: Close Up of Mechanical Alternative

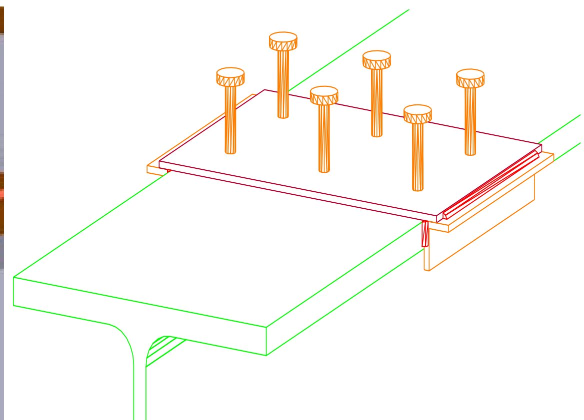

Figure 3-54: Alternative View

3.3 Pre-cast Option

When it was determined the WEBAC epoxy was not providing the desired results, a search for an alternative resulted in Dexter Hysol's 9460 epoxy. The epoxy is a modified epoxy adhesive that attains structural properties after room temperature cure. The two-part adhesive combines high peel strength with excellent shear strength. The bonds are permanently flexible and resist water, salt spray and many common industrial fluids. The goal of the search was to determine the feasibility of using epoxy as a shear transfer mechanism regardless of construction method. Consultations with epoxy manufactures led to the selection of the epoxy. The 9460 epoxy has been used in prior structural applications including pier retrofits on I-

80 in Sacramento. This epoxy was too viscous too allow pumping as was being explored. However, its high strength and good mechanical properties indicated this was a good candidate material if a pre-cast type operation was employed where access to the joining surfaces is available for gluing.

3.3.1 Epoxy Evaluative Testing

The first step was to evaluate the product and determine the optimum preparation procedures. A series of four tests was performed to make these determinations.

As the WEBAC epoxy appeared to have difficulty adhering to steel, tests whereby the new epoxy was applied solely to the steel were performed first.

The ultimate goal of these tests was to determine what sort of surface preparation would need to be performed for the construction of the prototype beam. For this reason, only a very small number of specimens were tested. Due to the high variability of the materials and the small scale of the specimens, a large number of specimens would have to have been tested to obtain statistically relevant measures of strength.

Test #1 Steel Lap Test

The goal of this test was to evaluate the adhesion of the epoxy to steel. Two specimens were tested with a contact area of 2"×4" per lap. One of these specimens can be seen mounted in the test frame in Figure 3-55.

Figure 3-55: Test Setup

Preparation of the steel surface was similar to that used when attaching glue mounted strain gages. The surface is ground to remove scale and debris. Next, degreaser is applied to remove contaminants. An acid etch is then applied to the surface and finally, a basic solution is used to neutralize the acid. Although this procedure was extreme and may be difficult to replicate in a fabrication environment, the goal was to obtain an upper bound on the strength.

Test Observations and Results

The results of the test were quite positive. The failure loads were 33.5 kips and 22.2 kips. Figure 3-56 shows the failure surface after testing. The average shear strength obtained from the testing of the two specimens was 3469 psi. This compares well with the published expected value of 3200 psi. The strength is 3.7 times that obtained from bare steel on steel tests utilizing the WEBAC epoxy. It is also 2.3 times the maximum value obtained using the WEBAC epoxy when the surface had been milled which was the preparation resulting in the highest strength for the WEBAC epoxy.

Figure 3-56: Close-up of failure

Test #2 Initial Concrete Block Push Test

The next step in assessing the strength of the epoxy was its adherence to concrete. To determine the concrete bond strength, specimens as shown in Figure 3-57 were fabricated. Each specimen was composed of two identical concrete blocks with a strip of steel sandwiched between. The contact area was 4"×4" for the specimens. The strip of steel is offset from the concrete blocks, allowing it to be pushed relative to the blocks. The bases of the blocks are set in Hydrostone, to eliminate stress concentrations due to surface irregularities.

Figure 3-57: Test Setup

Three identical specimens were fabricated and tested. It was obvious that the failure of the first specimen was not due to shearing. A free body diagram of the setup reveals a transverse tensile component required to resist the moment developed due to the offset of the steel plate from the face of the block.

Transverse restraint was provided during the testing of the second specimen through the use of a C-clamp as shown in Figure 3-58. Again, inspection of the failure indicated the failure was not due entirely to shear. It was determined the C-clamp was not stiff enough to prevent movement.

Figure 3-58: Test Setup with C-Clamps

The goal of the clamping device is not to apply pre-force to the system. Rather, the goal is to provide a resistance with enough stiffness such that lateral displacement is minimized during testing. Therefore, for the testing of the third specimen, a cage utilizing channels and 5/8" threaded rod was fabricated as shown in Figure 3-59. It is important to note that the cage was not overly tightened and installed only snug tight.

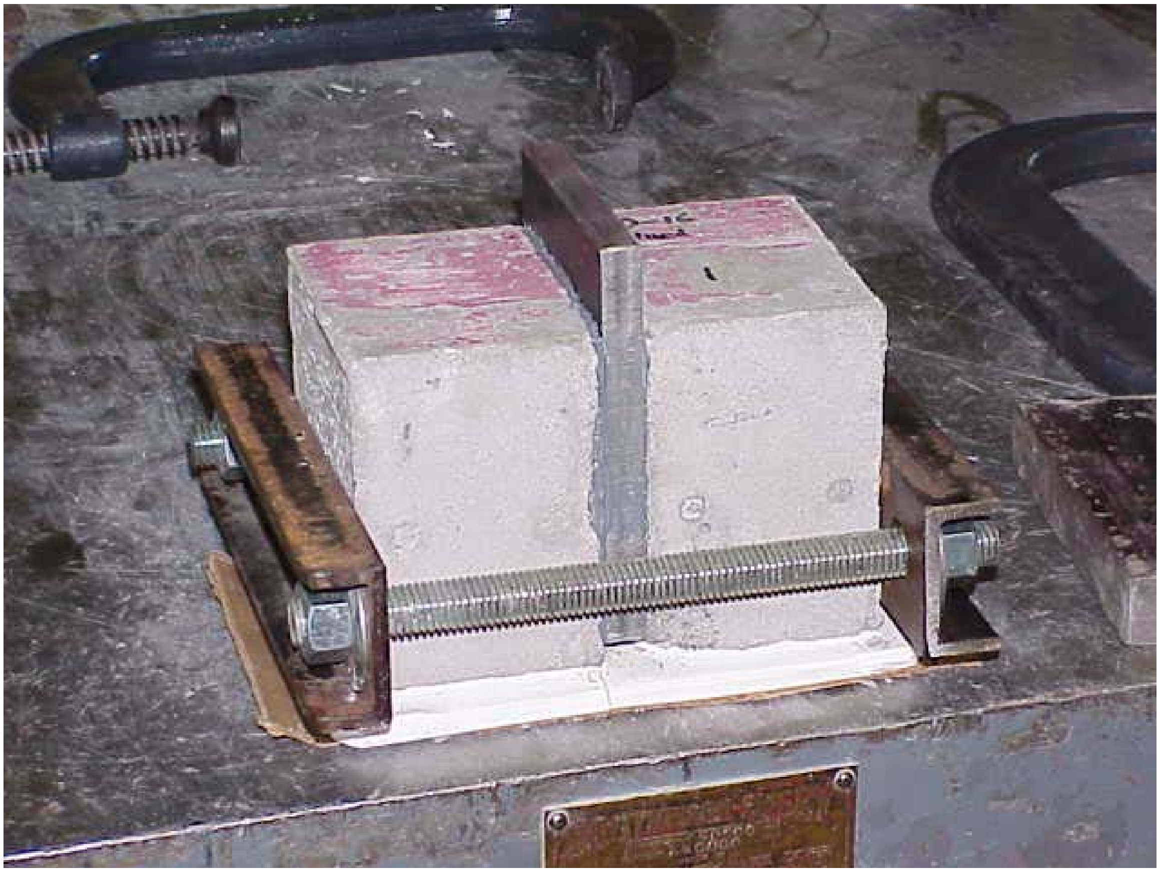

Figure 3-59: Test Setup with lateral restraint yoke

Test Observations and Results

It appeared in all tests that the concrete was exhibiting localized failure. In Figure 3-60, one can see a large amount of concrete remaining on the steel. This was the first specimen tested and did not have any lateral restraint.

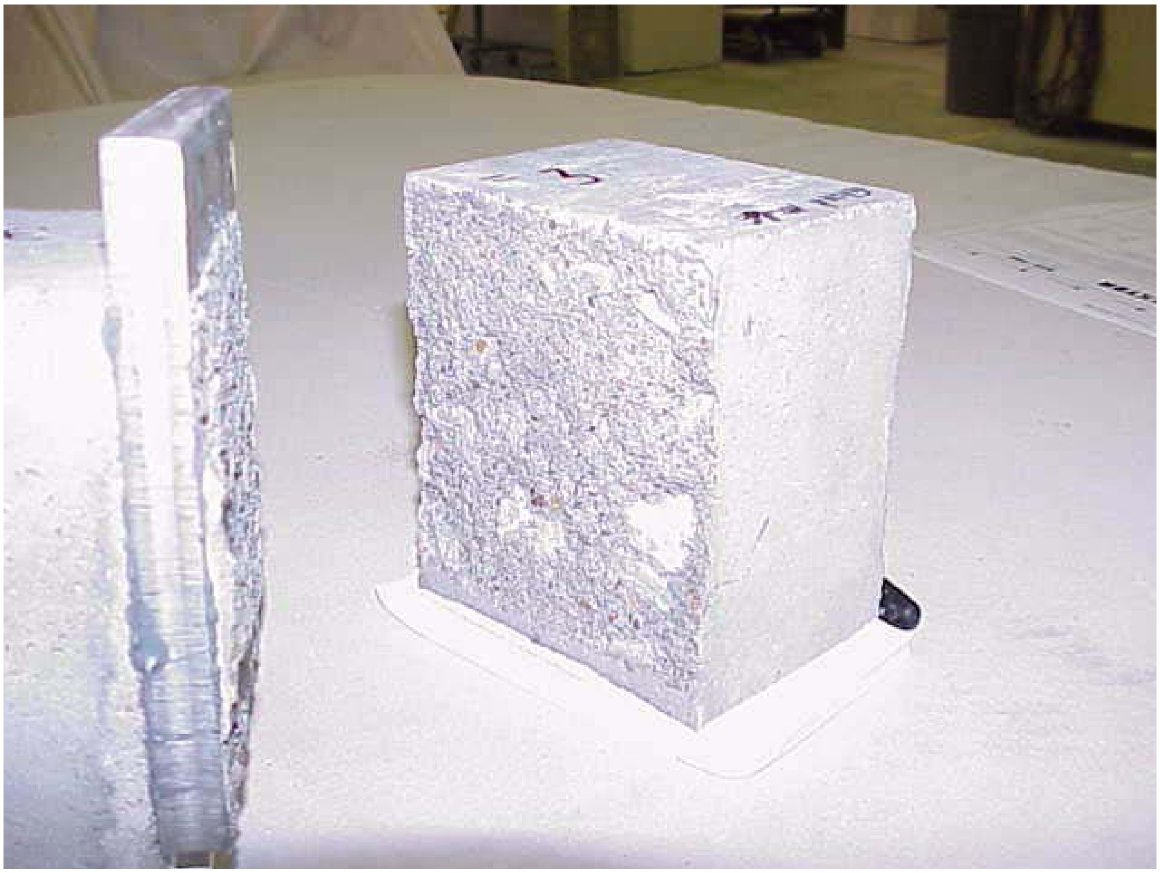

Figure 3-60: Close-up of failure first specimen

Figure 3-61 shows the second specimen which used a C-clamp to provide lateral restraint. Here, one can see there is much less concrete left on the steel.

Figure 3-61: Close-up of failure second specimen

Finally, in Figure 3-62 is shown the final specimen which had a rigid lateral restraining cage. One can see very little concrete remaining on the steel. This indicates a true shear mode of failure. Although it is not entirely evident in the picture, the failure surface is composed of very fine concrete and cement particles embedded in the epoxy, indicating the concrete as the primary failure element. There are a few locations where the epoxy de-bonded from the steel.

Figure 3-62: Close-up of failure third specimen

The following table indicates the ultimate shear strengths obtained from the tests. Although the restrained specimens displayed lower shear strength, this was attributed to the variability of other factors. The improvement of the failure mode was the desired result and a restraining device was used in subsequent testing.

Table 3-1: Initial Push Test Results

Specimen	Restraint	Load (Kips)	Stress (psi)
1	None	18.6	581
2	C-Clamp	13.4	419
3	Yoke	12.0	375

TEST #3 ADVANCED SURFACE PREP CONCRETE BLOCK PUSH TEST

Based on the results of the first two tests, it was determined that the concrete was the limiting factor and therefore, a series of specimens was tested attempting to determine the optimum surface prep to obtain the greatest adhesion to the concrete.

Three specimens were tested with a contact area of 2"×4". The first was prepared by removing the fines from the surface of the specimen with grinder fitted with a wire wheel brush. Unfortunately, use of the wire brush appeared to leave a residue on the concrete. This residue was then removed using an acid wash process similar to the preparation used on the steel. Figure 3-63 shows the results of each step. The block on the right has not been touched. The middle block shows the residue after brushing. The block on the left shows the removal of the residue.

Figure 3-63: Surface preparation steps

The second specimen was cut with a concrete saw. The goal was to reveal large faces of competent aggregate for the epoxy to adhere to, eliminating the fine particle interface. Again, this was done to try and determine an upper bound to the expected strength and would not be a practical preparation method.

The third specimen was used as a control and therefore had no special surface preparation applied to it.

Test Observations and Results

Figure 3-64 shows the control block after testing. The failure is similar to the third specimen from test #2.

Figure 3-64: Close-up of failure control specimen

Figure 3-65 shows the block which had been prepared with the wire wheel after testing. This specimen did not perform as well as the control. This is most likely due to the presence of the residue on the concrete.

Figure 3-65: Close-up of failure wire prepped specimen

The cut block is shown in Figure 3-66. The resulting strength from this specimen was 2038 psi. This is an 23% increase over the control specimen.

Figure 3-66: Close-up of failure cut specimen

The results from the tests are summarized in the following table.

Table 3-2: Surface Prep Push Test Results

Specimen	Surface Prep	Load (Kips)	Stress (psi)
1	None	26.6	1663
2	Wire	18.4	1150
3	Cut	32.6	2038

TEST#4 GROUND CONCRETE BLOCK PUSH TEST

The previous test was designed to provide an upper bound on the expected shear strength. However, the preparation methods were either too costly or impossible to actually be implemented. Therefore one final set of tests was performed to determine which minimal surface preparation method resulted in the best performance.

Of the three blocks tested, the first was prepared with a sanding disk, the second with a grinding wheel, and the final was left as a control.

Based on the results of these tests, it was determined that none of the minimal surface preparation methods resulted in a substantial increase in shear strength. Therefore it was determined that for the prototype beam test no additional surface preparation, other than removing free surface particulates, would be performed.

3.3.2 Prototype Beam Test

To further investigate the use of epoxy as a shear transfer mechanism, a prototype beam utilizing epoxy was designed, built, and tested.

Specimen Design

Several criteria governed the design of the specimen. First, since this was an initial feasibility test, the specimen was to be small, economical, and easy to test. Second, to maximize the load on the shear transfer mechanism, the specimen was designed such that the neutral axis was located near the top of the steel section at the ultimate load level. Finally, there was a desire to observe the behavior of the system at large deformation levels. The large amount of inelastic deformation was desired to evaluate the ductility of the shear transfer mechanism.

The selected specimen consisted of a 10 foot long, 100 inches between supports, W8×21 Grade 50 steel beam topped by a 25 inch wide by 5-1/4 inch thick concrete slab with specified concrete compressive strength of 5000 psi. A 3/8"×5-1/2" plate was welded to both sides of the web to increase the shear capacity. Bearing stiffeners were added at the supports and under the load. Slab reinforcement consisted of a top and bottom layer of #4 bars (5 inches on center) in both the longitudinal and transverse directions. The transverse bars were positioned closest to the slab face with one inch of cover to the bottom and 3/4" of cover to the top. Figure 3-67 illustrates the cross section of the test specimen.

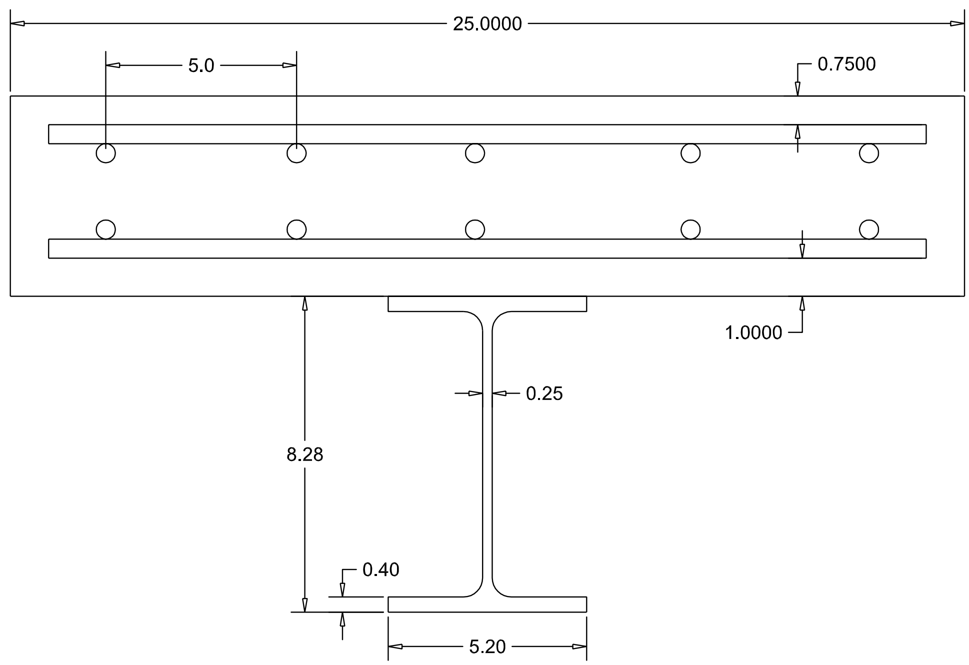

Figure 3-67: Test Specimen Dimensions

A strip of 24 gauge steel five inches wide was placed in the bottom of the form the length of the beam. This provided a very smooth and level surface for the epoxy to bond against.

The predicted moment at which the bottom flange would begin to yield assuming fully elastic composite behavior was determined to be 2,450 kip-in corresponding to a simply supported load of 98 kips. This was expected to generate 1000 psi of shear stress at the level of the connection.

The only preparation of the concrete surface was a light brushing and water rinse. The surface was allowed to dry overnight and compressed air was used to remove any dust which may have accumulated.

The steel surface was prepared by grinding to bare steel and then degreasing. A mild acid etch was then applied to further clean the surface. The acid was then neutralized with a base wash. Figure 3-68 shows the preparation process.

Figure 3-68: Preparation of Steel Surface

The epoxy was then mixed and applied generously to the steel surface. After the slab had been lowered on the steel, the slab was clamped to the beam. This clamping squeezed out the excess epoxy, leaving a minimal glue line. This excess epoxy can be seen in Figure 3-69 of the assembled system.

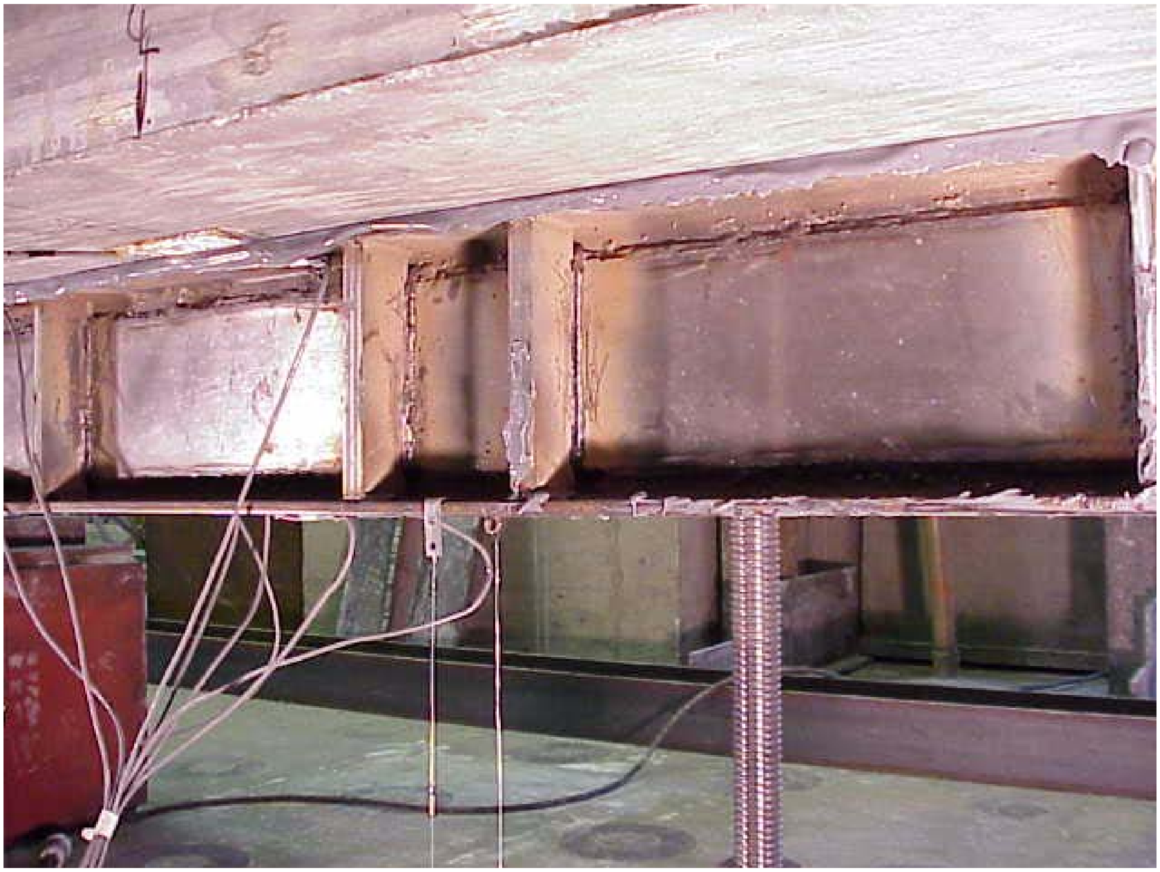

Figure 3-69: Assembled Epoxy Beam Specimen

TEST SETUP AND INSTRUMENTATION

The beam was tested in three-point bending with a concentrated load at midspan. The specimen was supported by rollers at the ends to allow free rotation. The roller base at one end was placed on Teflon to allow free longitudinal translation. The end detail can be seen in Figure 3-70. Load was applied using hydraulic rams which pulled the specimen against the strong floor of the laboratory. The completed setup can be seen in Figure 3-71.

Figure 3-70: Close-up of Epoxy Beam Specimen End Support

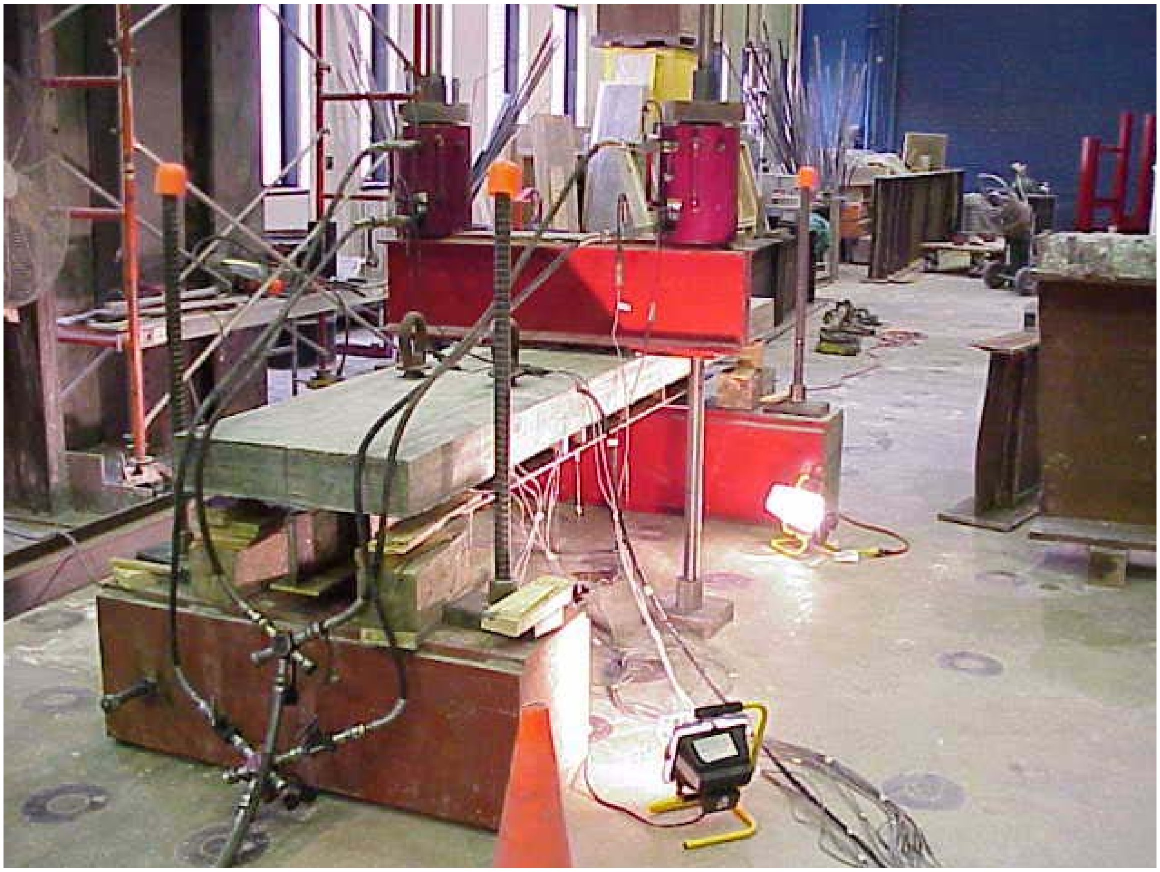

Figure 3-71: Loading Setup

Linear displacement potentiometers were placed at beam quarter points to monitor the deflection. Glue bondable strain gages were applied to the steel beam and surface of the deck to monitor the strains developed during the loading. Vibrating wire gages were used to monitor the strains inside the deck. The amount of instrumentation was kept low to minimize cost. Therefore, instrumentation was focused on the primary area of interest, which is the strain profile near midspan.

TEST PROCEDURE AND RESULTS

The loading was applied in small increments, or load stages. At each load stage, data was collected from the instrumentation using an acquisition system. Figure 3-72 is a plot of the applied load versus the midspan deflection. As can be seen in the figure, the path was very linear up until load stage 8. During the next increment of loading a large bang was heard and the load dropped considerably. Examination of the specimen revealed that the concrete deck on the west side had broken free from the steel girder. In interest of completeness, loading was continued until the east side failed as well which occurred a short time later.

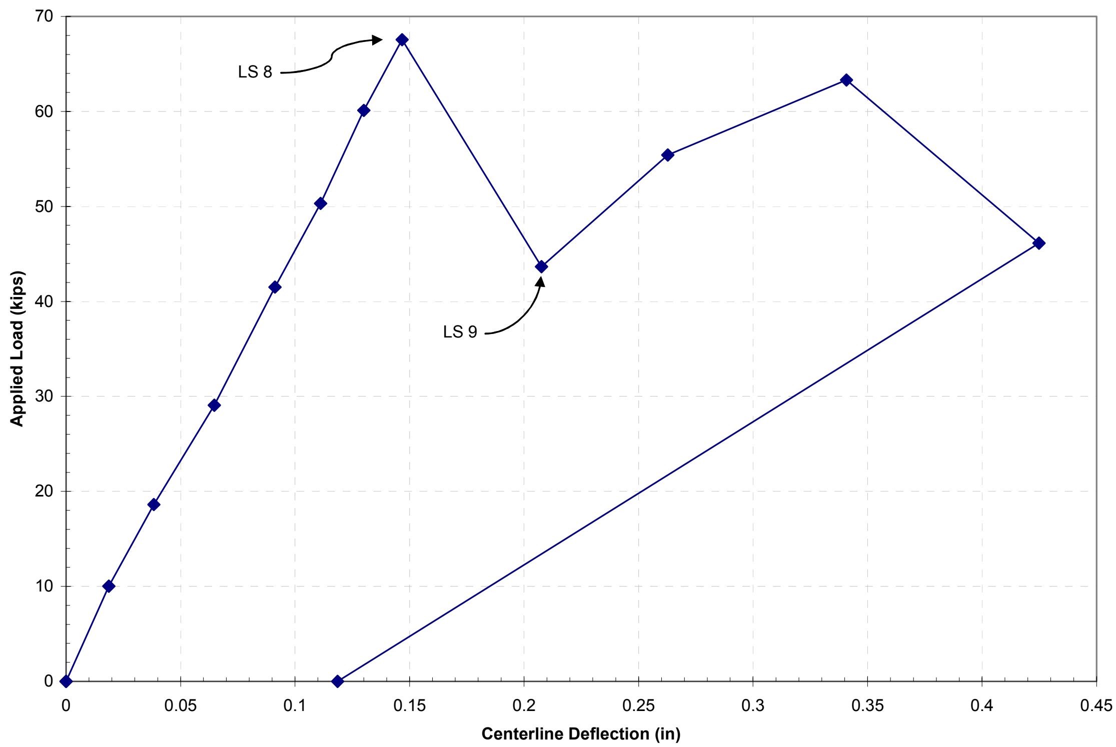

Figure 3-72: Load Deflection Curve

After the load was removed, it was discovered that the entire slab was completely separated from the steel beam and could be simply lifted away. The underside of the concrete slab can be seen in Figure 3-73.

Figure 3-73: Underside of Deck after removal

Examination of the beam revealed that the concrete to epoxy bond had been the weak link as indicated by the fact that all of the epoxy remained on the steel. Figure 3-74 shows a close up of this surface. Although it may not be clear from the figure, it was quite evident that a fine residue of cement was embedded in the epoxy. Therefore, it does not appear as though the epoxy failed, but rather the face of the concrete pulled away.

Figure 3-74: Exposed Epoxy Surface

One exception to the overall mode of failure was near midspan. Figure 3-75 shows a close-up of this region. It would appear as though the downward force from the applied load prevented separation of the concrete from the epoxy and there was some localized shear failure of the epoxy itself.

Figure 3-75: Failure Surface Near Mid-Span

Figure 3-76 shows the strain profile obtained from the load stages just before and just after the initial failure (load stages 8 and 9, respectively). Just prior to the failure, the strain profile is linear through the entire depth of the section indicating good composite action. In fact, there is no slippage at the interface as is often observed in beams constructed utilizing shear studs. At load stage 9, just after failure, the slippage is quite obvious although there still appears to be some interaction remaining. This remaining interaction is due to the fact that only one side has de-bonded and the other side remains composite.

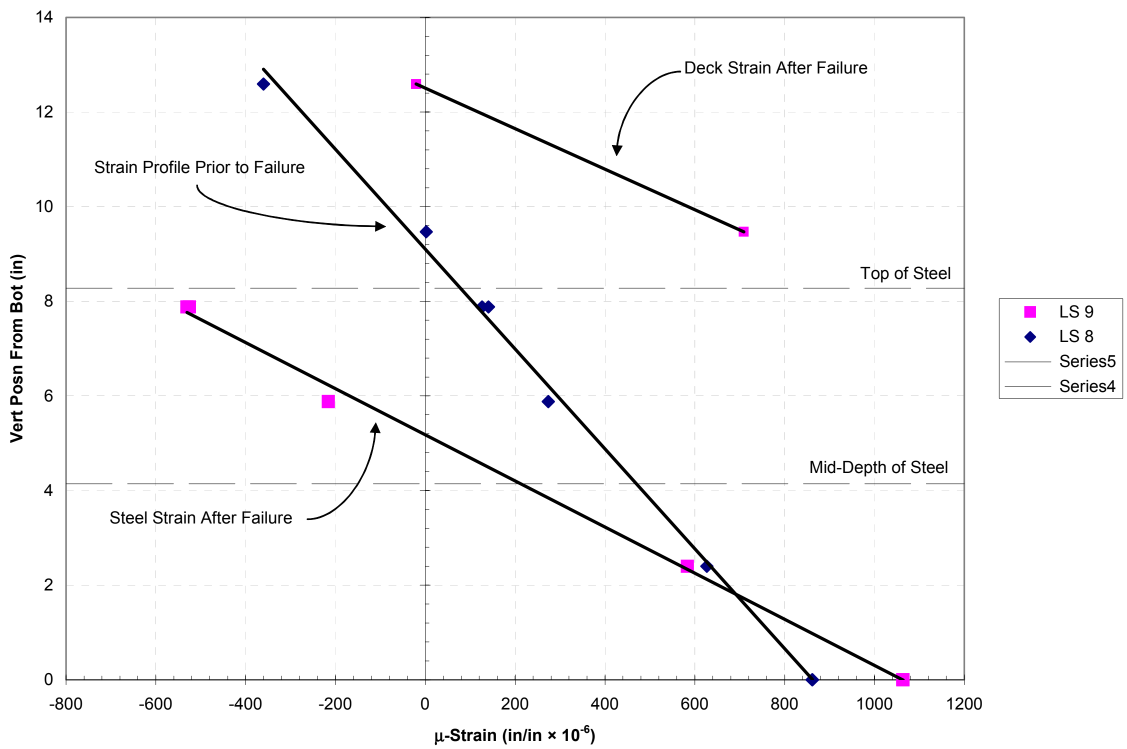

Figure 3-76: Resulting Strain Profiles

After the loading of the beam was continued through failure of the second side, it is evident that the steel beam is acting completely independently of the concrete slab. This can be seen in Figure 3-77 which is the strain profile at midspan after the second side has failed. It can be seen that the strain profile in the steel section passes through the mid-depth of the beam. If there was any remaining interaction with the concrete slab, the neutral axis would be above mid-depth.

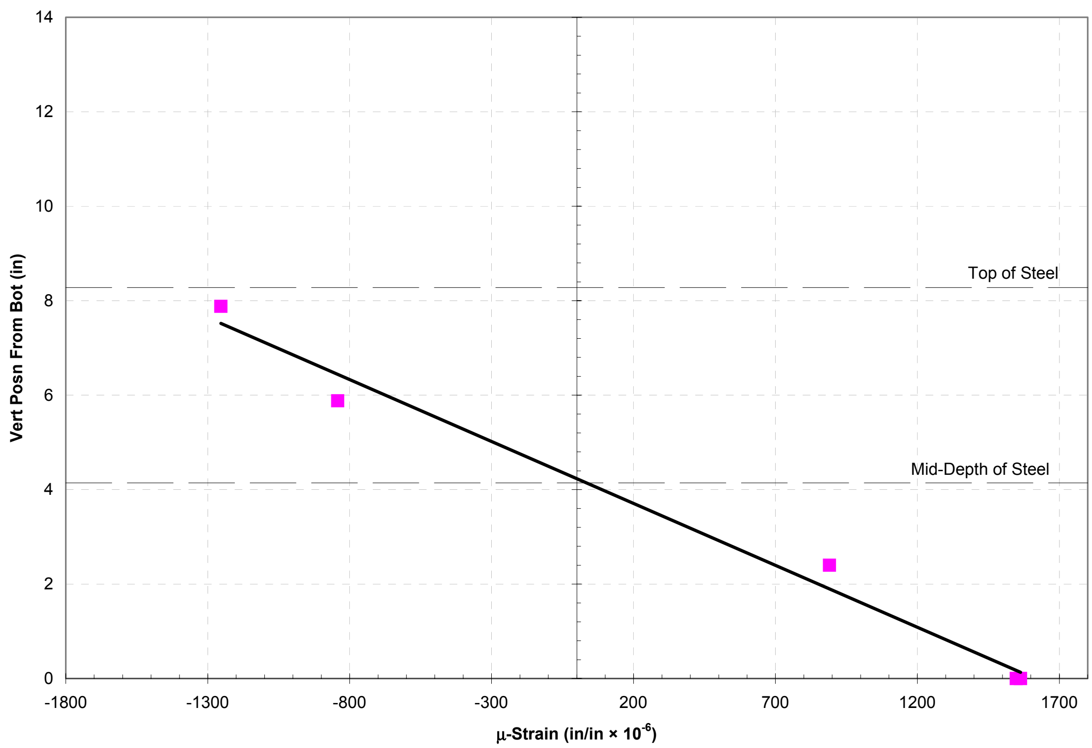

Figure 3-77: Strain Profile after Failure

CONCLUSIONS FROM TESTING PROTOTYPE BEAM

Based on the results of the prototype beam test, it was determined that any feasible system cannot rely on epoxy alone to transfer the shear force. Not only was the strength well under the yield strength, the failure was sudden and complete.

There was no deviation from linearity prior to failure to signal an overload condition. When the failure occurred, there was a very large drop in capacity with complete de-bonding occurring and no display of ductility until the steel beam alone was carrying the entire load. These are all considered highly undesirable properties in structural design and any system displaying such should be avoided.

3.4 STUD STRIP

Figures 3-78 through 3-89 show the fabrication process utilizing stud strips, beginning with the bare steel girder as shown in Figure 3-78.

An extruded plastic coping is applied to the sides of the flange as shown in Figure 3-79. Figure 3-80 shows a close up of the coping. The vertical projection, when embedded in the concrete deck, will act as a bead of caulking in preventing the epoxy (to be injected later) from seeping out from the sides of the flange. Since the effectiveness of the coping comes from its embedment into the concrete, the coping can be attached in whatever manner is simplest, such as contact cement or other adhesive.

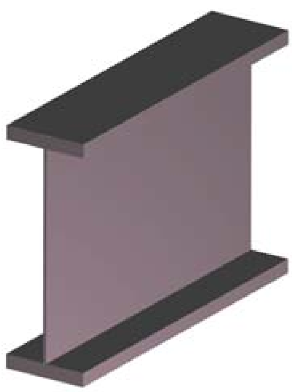

Figure 3-78: Bare Steel

Figure 3-79: Coping Applied

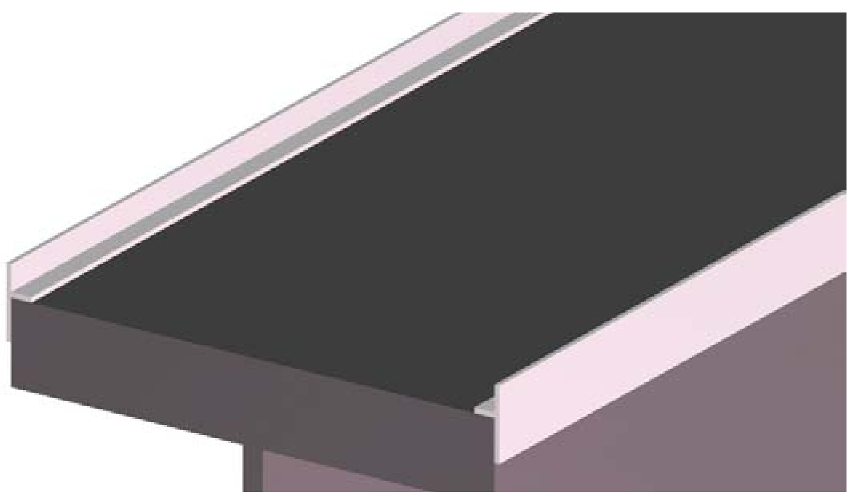

Figure 3-80: Close-up of Coping

The next step is the placement of the stud strips as shown in Figures 3-81 and 3-82.

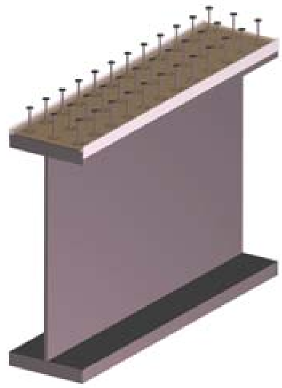

Figure 3-81: Stud Strip in Place

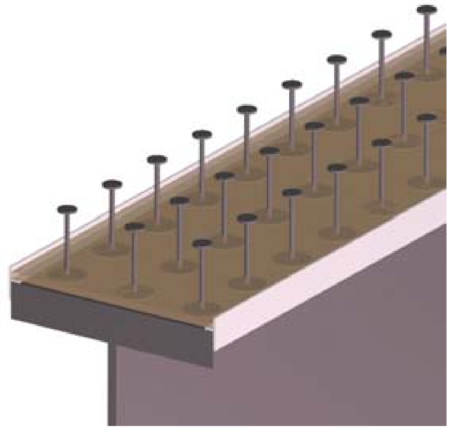

Figure 3-82: Close-Up of Stud Strip

Figure 3-83 is a close-up of the stud strip. The strip is composed of a low stiffness membrane or substrate in which double headed studs are embedded. One possible material for the membrane would be fiber reinforced plastic. Use of fiber reinforced plastic would allow the studs to be incorporated during the layup of the material with the strands of fiber being passed around the studs.

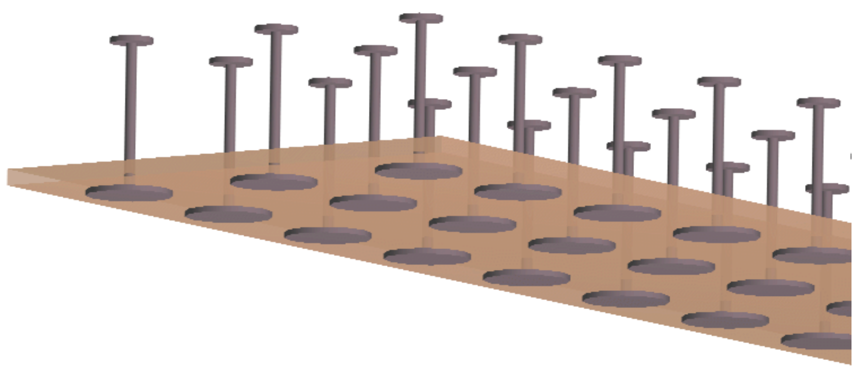

Figure 3-83: Underside of Stud Strip

Epoxy injection tubes would then be placed in the stud strips. These would extend to the top of the deck, allowing epoxy injection after the deck has been cast. The tube could be part of the manufactured strips, or be placed in pre-drilled holes.

Figure 3-84 shows the system ready for the deck to be cast, which is shown in Figure 3-85.

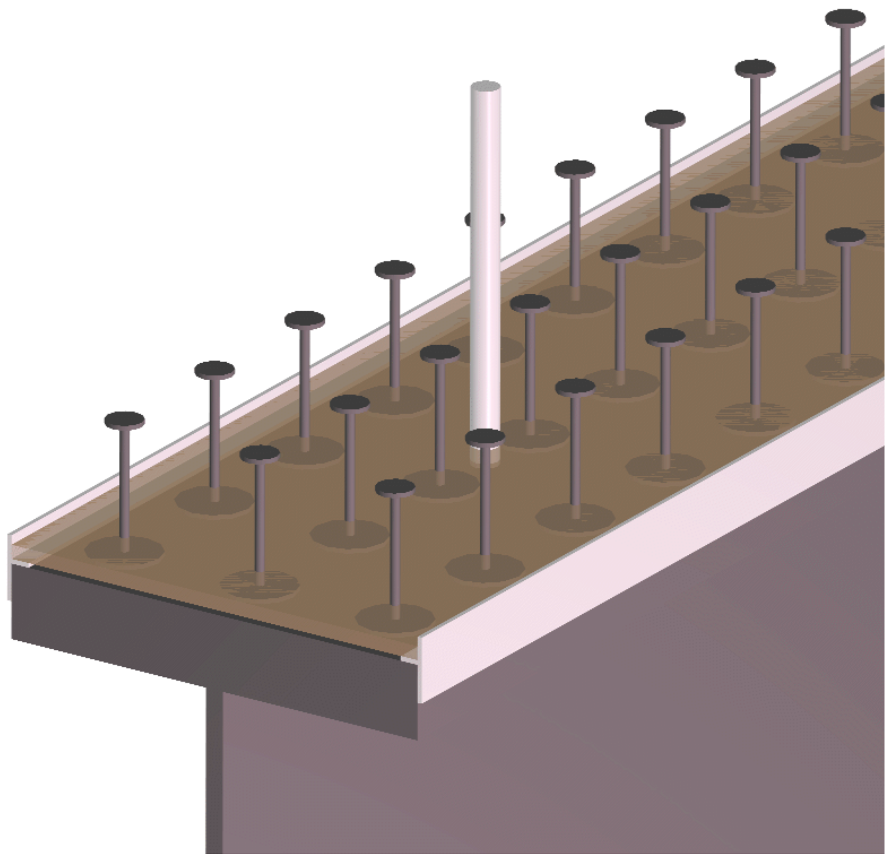

Figure 3-84: Epoxy Injection Tubes in Place

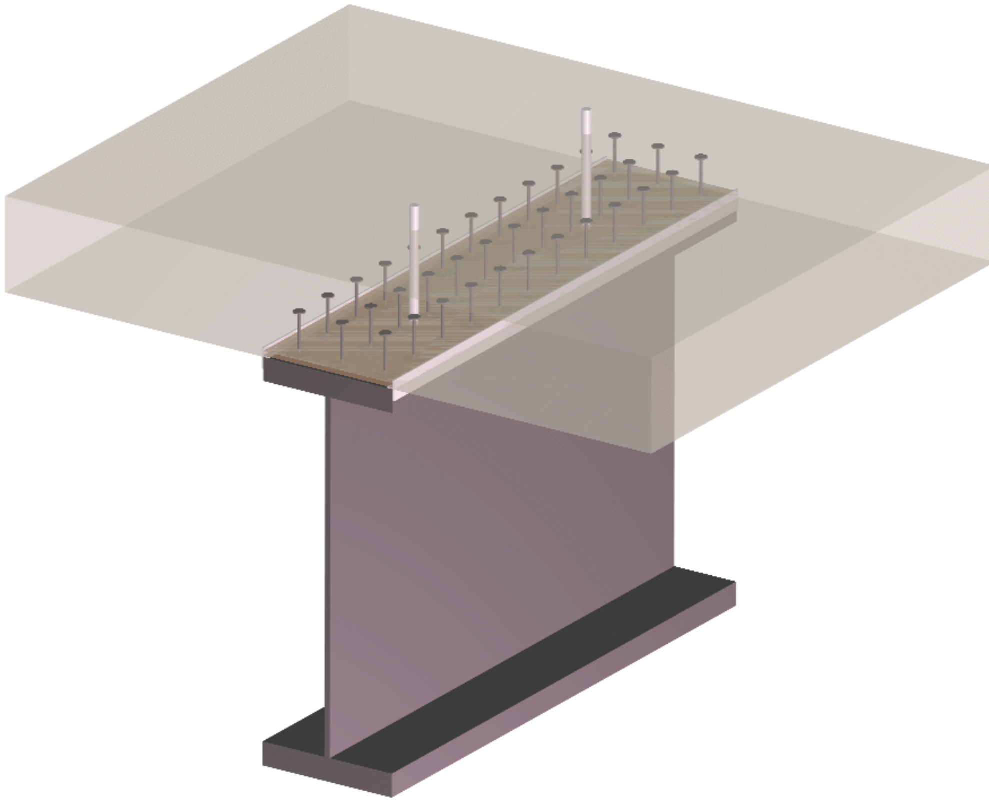

Figure 3-85: Deck Cast

Once the deck has cured and been allowed adequate time to shrink, the epoxy is injected through the holes into the void which exists between the top of the girder and the stud strip as shown in Figures 3-86 and 3-87.

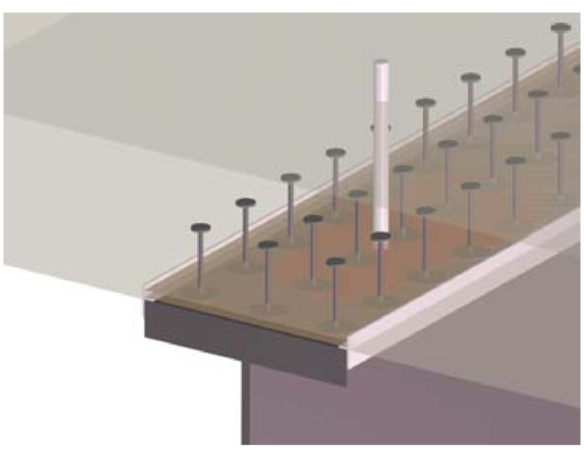

Figure 3-86: Begin Pumping Epoxy

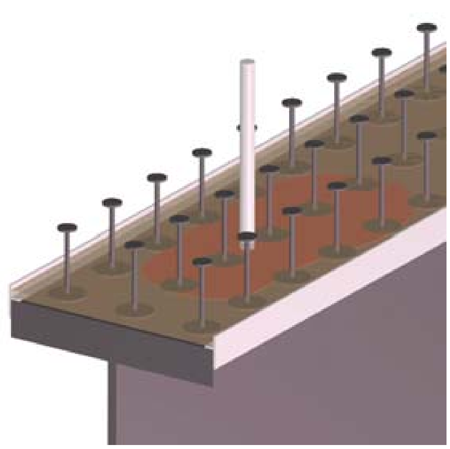

Figure 3-87: Deck Removed for Clarity

Figures 3-88 and 3-89 show the completed system.

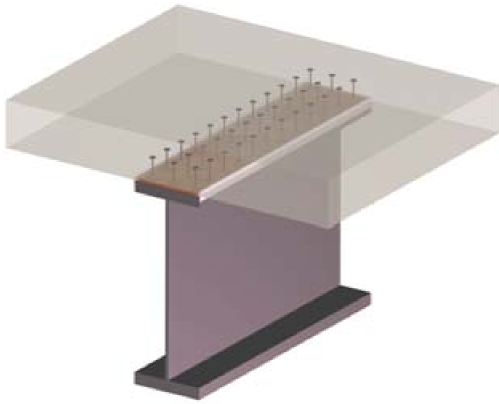

Figure 3-88: Final Stud Strip System

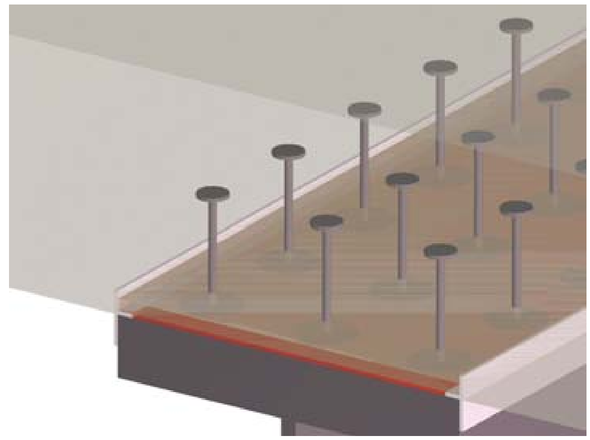

Figure 3-89: Close-Up of Stud Strip System

This system improves on the plain epoxy injection system in a couple of ways. First, there is a mechanical interlock between the stud strip and the concrete. Therefore, the system does not rely on the adhesion of the epoxy to the concrete surface. Second, the larger void allows the use of higher viscosity epoxy which was found to perform better when joining dissimilar materials. Finally, flexibility in choosing the stud strip substrate would allow the selection of a material which would perform well with the chosen epoxy which was found to bond well to the steel.

3.5 Epoxy embedded studs

The concept behind the epoxy embedded studs alternative is to delay embedment of the shear studs until after concrete shrinkage has occurred. A similar concept has also seen use in pre-cast systems (Price 2000). The construction steps of this alternative are illustrated in Figures 3-90 through 3-98.

Figure 3-90 shows the shear studs in place on the steel flange. This alternative does not directly address the tripping hazard posed by the shear studs. Therefore, these studs may be field or shop applied depending on the requirements.

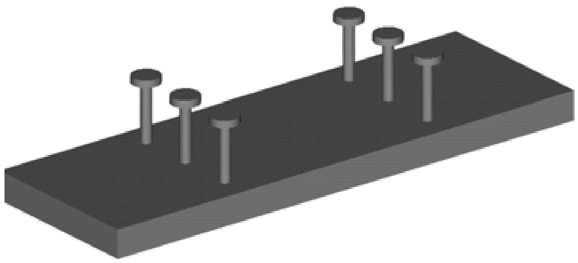

Figure 3-90: Studs on Top of Girder

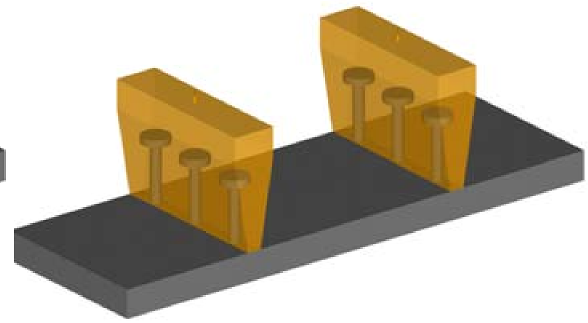

Figure 3-91: Boot in Place

The second step is the placement of a plastic boot over the studs as shown in Figure 3-91. The exact design of these boots is not finalized. Several details have been considered. Fins on the inside of the boot can be used to secure the boots to the studs. Adhesive applied to the bottom of the boot can also help to secure the boots in place and also create a seal preventing concrete seepage into the boot during deck casting.

Notice the locator tag formed into the top of the boot. These are very commonly found with embedment devices. The locator protrudes from the top of the concrete which allows the height of the boot to be slightly less than the depth of the deck preventing interference during deck placement and finishing as shown in Figure 3-92.

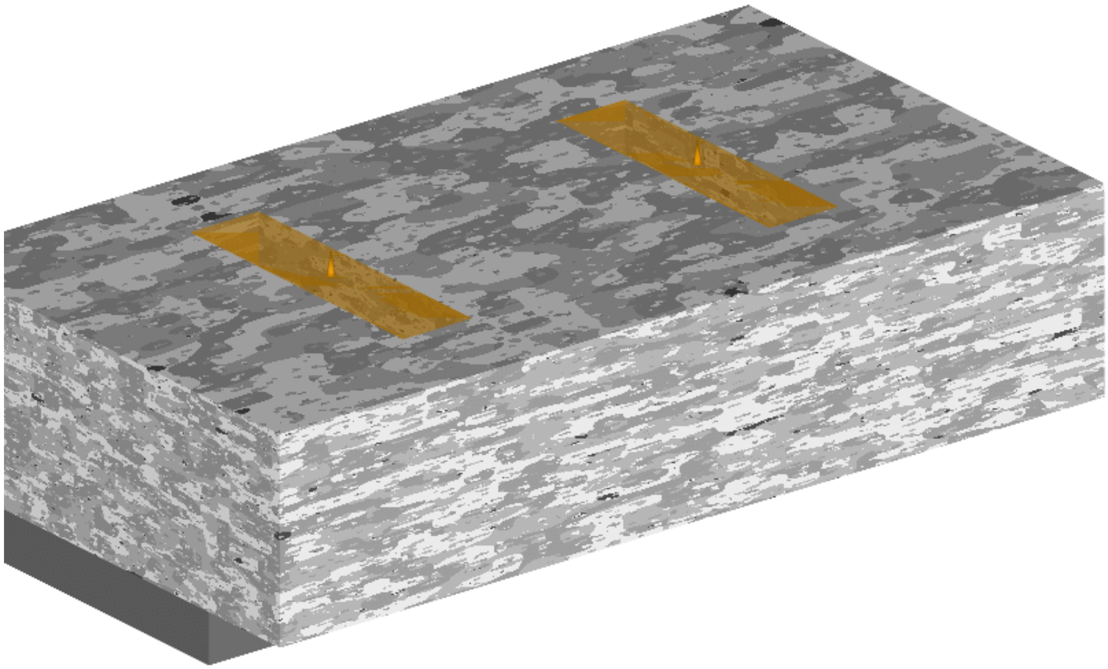

Figure 3-92: Cast Deck

Once the concrete has cured, the boots can be located by the protruding tag. The boot is then pierced and compressed air is used to force the boot out leaving a hollow cavity around the shear studs as shown in Figures 3-93 and 3-94.

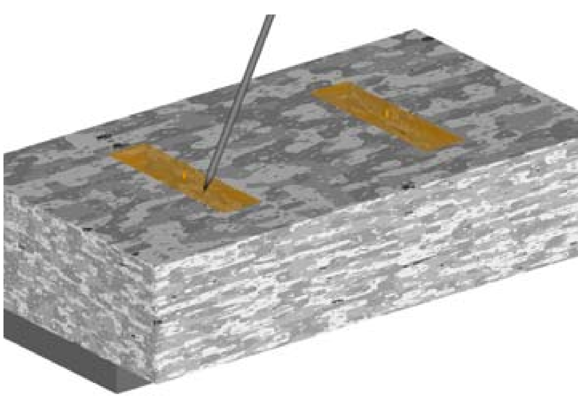

Figure 3-93: Pierce Boot

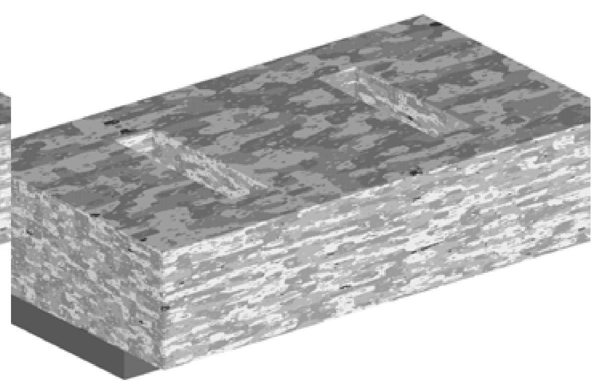

Figure 3-94: Boot Removed

These cavities are then grouted full as shown in Figures 3-95 and 3-96. Several options are under consideration for the grout including conventional Portland cement based grout, pure epoxy, or an epoxy mixed with fine aggregate such as sand to reduce the required amount of epoxy.

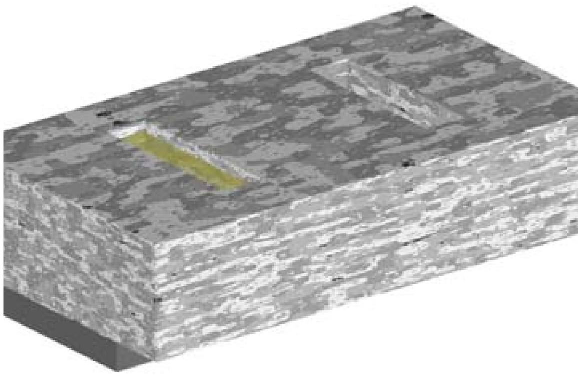

Figure 3-95: Grouting

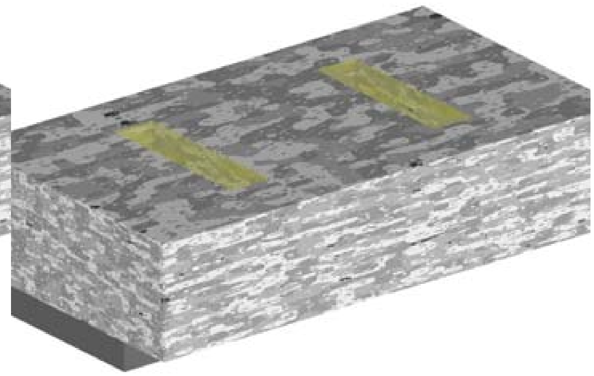

Figure 3-96: Finished Grouting

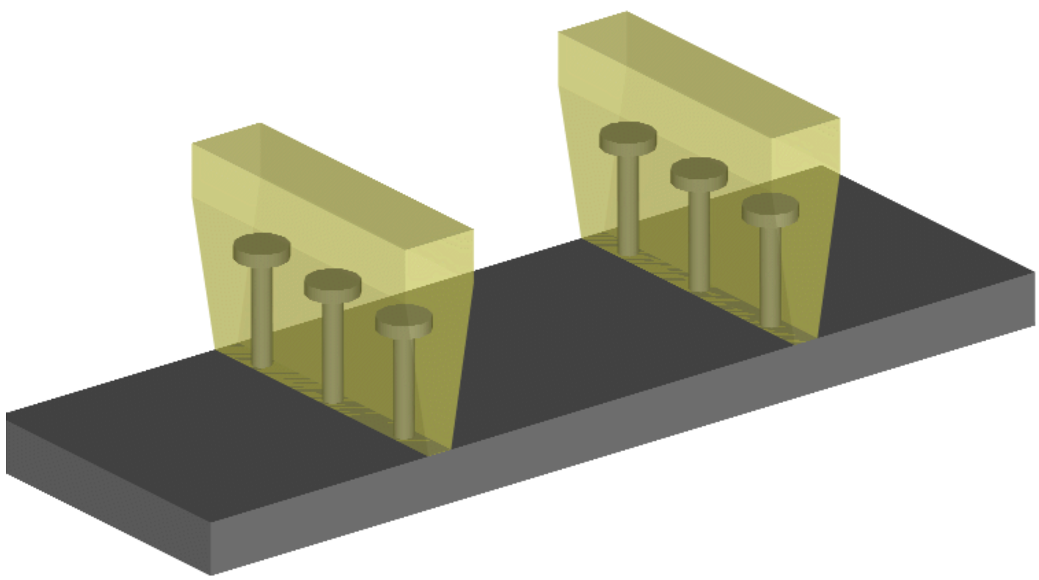

Figure 3-97: Resulting Grout Plug

The boots should be tapered for two reasons. The first reason for the taper is that the tapered boot facilitates its removal. Second, the taper creates a locking effect, as can be seen in Figure 3-97, resisting the vertical component of the shear force.

Figure 3-98 is an illustration showing the completed system.

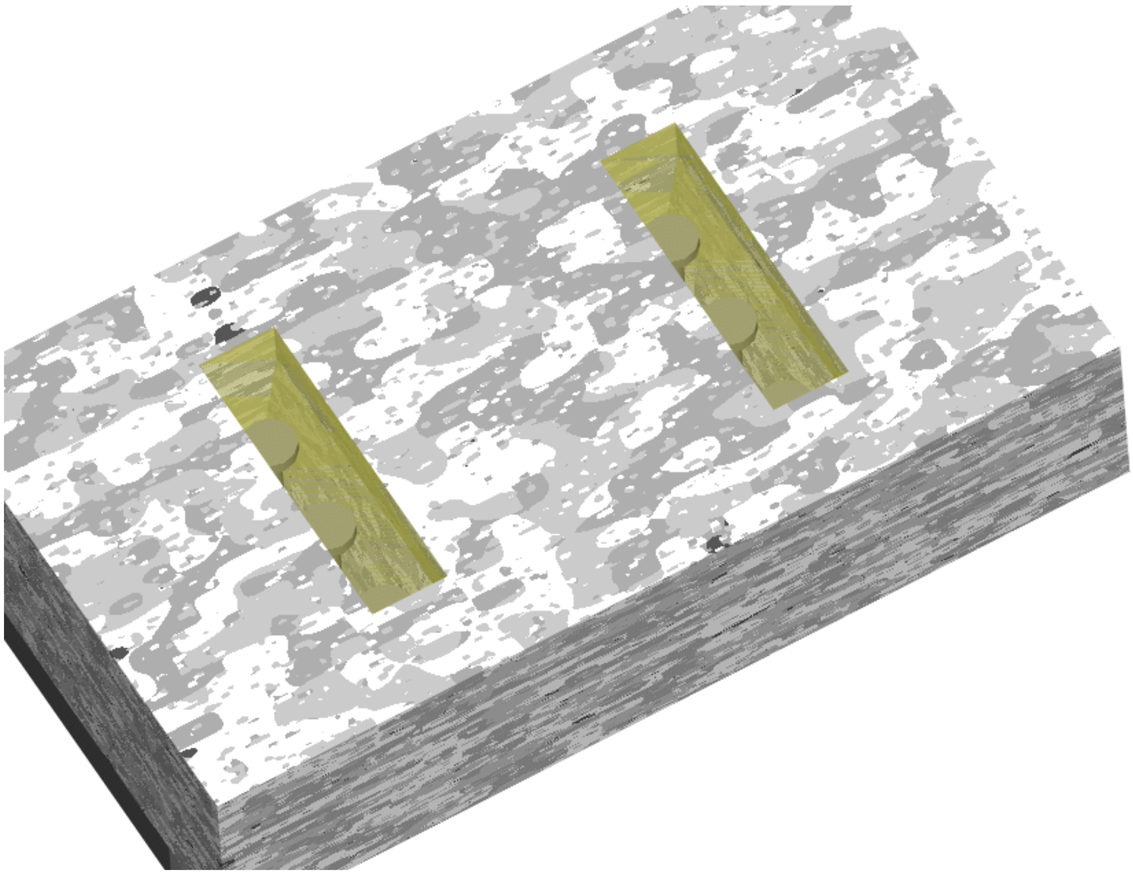

Figure 3-98: Completed System

The epoxy grouted stud system uses the epoxy in a mode which has been proven successful. Epoxy grouting is a very common practice for embedding rebar or hooks into concrete. This is essentially what the system is doing as well. Tapering of the hole further enhances the performance by creating a wedge shaped epoxy plug.

3.6 Mixed aggregate method

One idea was conceived which makes use of aggregate to improve on some of the problems observed with the pumped epoxy alternative. Figures 3-99 through 3-110 illustrate the concept.

The process begins similar to that used in the stud strip alternative. Plastic coping is placed on the bare steel beam as shown in figures 3-99 through

3-101. The vertical leg of the coping embeds into the concrete, thereby preventing leakage and guiding the epoxy along the length of the girder.

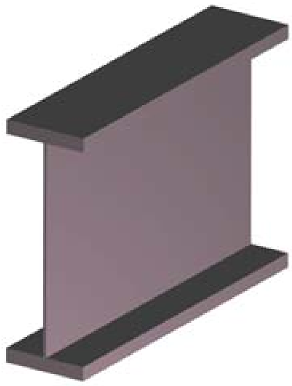

Figure 3-99: Bare Steel

Figure 3-100: Coping Applied

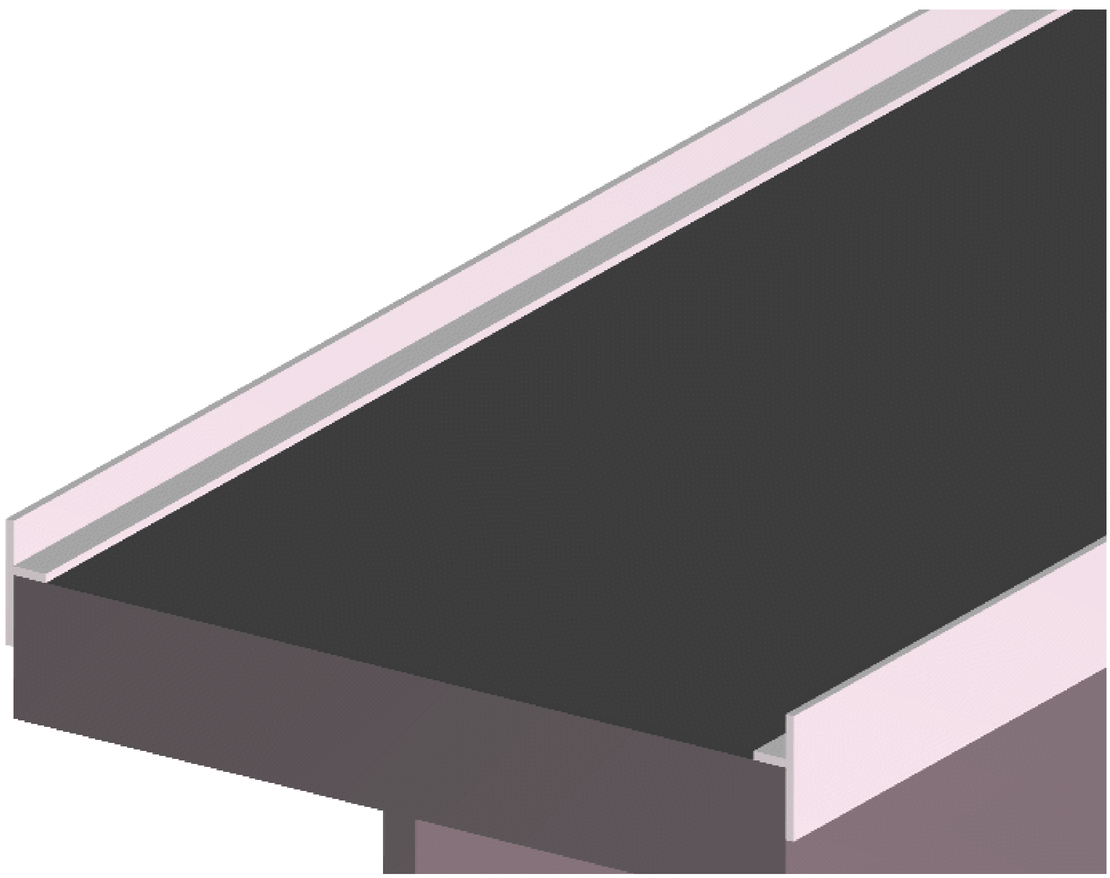

Figure 3-101: Close-up of Coping

The next step is the placement of a layer of sand covering the top as shown in Figures 3-102 and 3-103. The idea is that when the deck is cast, the concrete will infuse the layer of sand and create an irregular surface.

A tube is positioned in the sand (Figure 3-104) which will be used for injection of the epoxy after the deck has been cast (Figure 3-105).

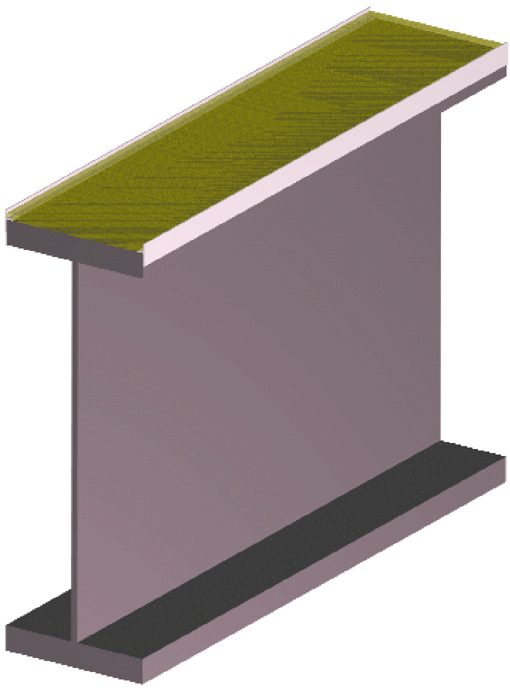

Figure 3-102: Layer of Sand

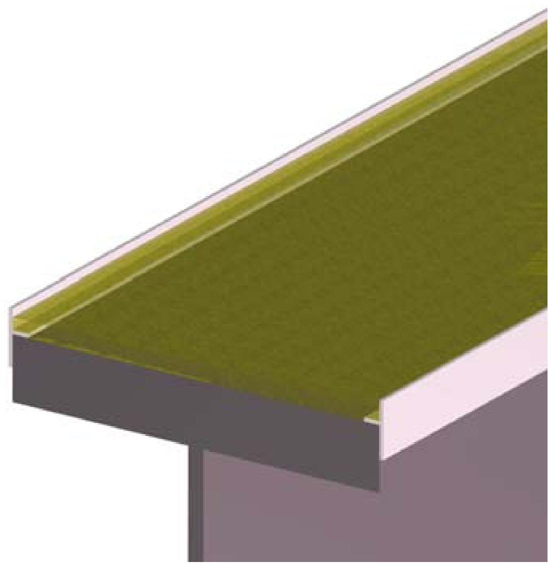

Figure 3-103: Close-Up of Sand Layer

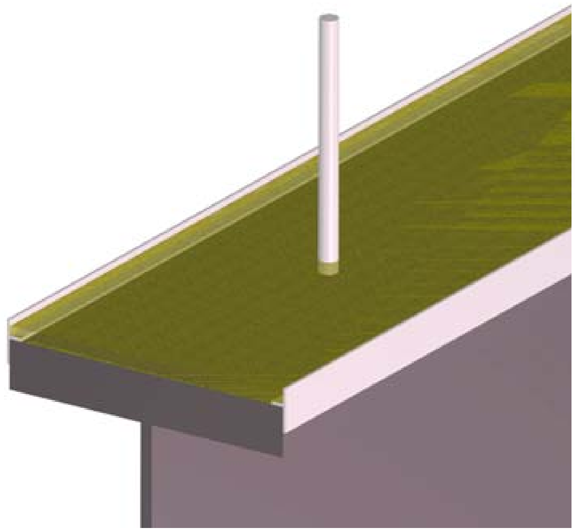

Figure 3-104: Injection Tube

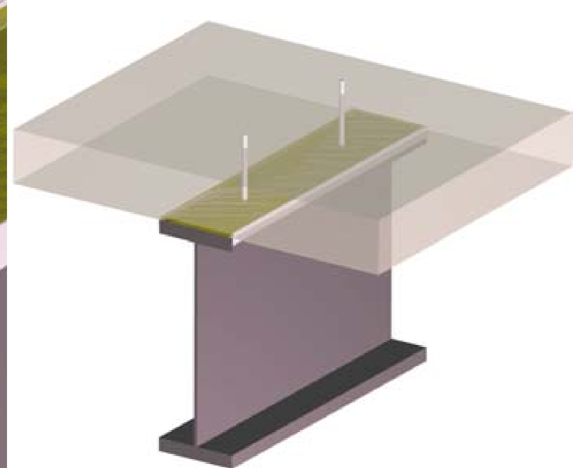

Figure 3-105: Cast Deck

After the concrete has cured and shrunk, the tubes must be located and the ends cleared. Next, a nipple is affixed to the end of the tube. Finally epoxy is injected through the layer of sand as shown in Figures 3-106 and 3-107.

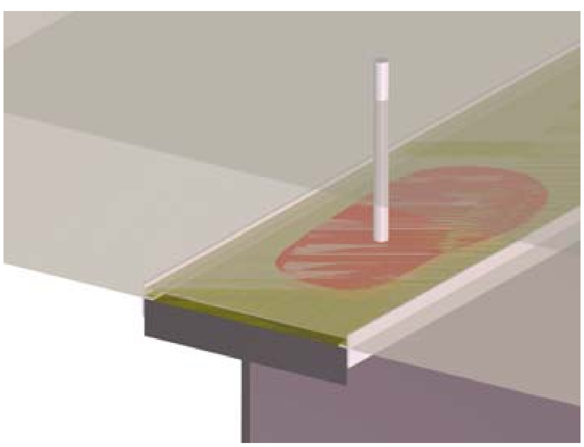

Figure 3-106: Pumping Epoxy

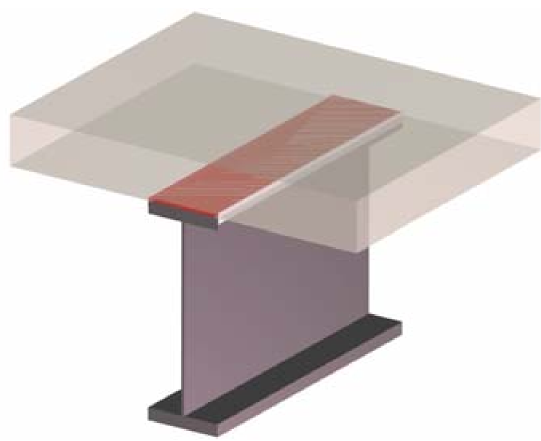

Figure 3-107: Final System

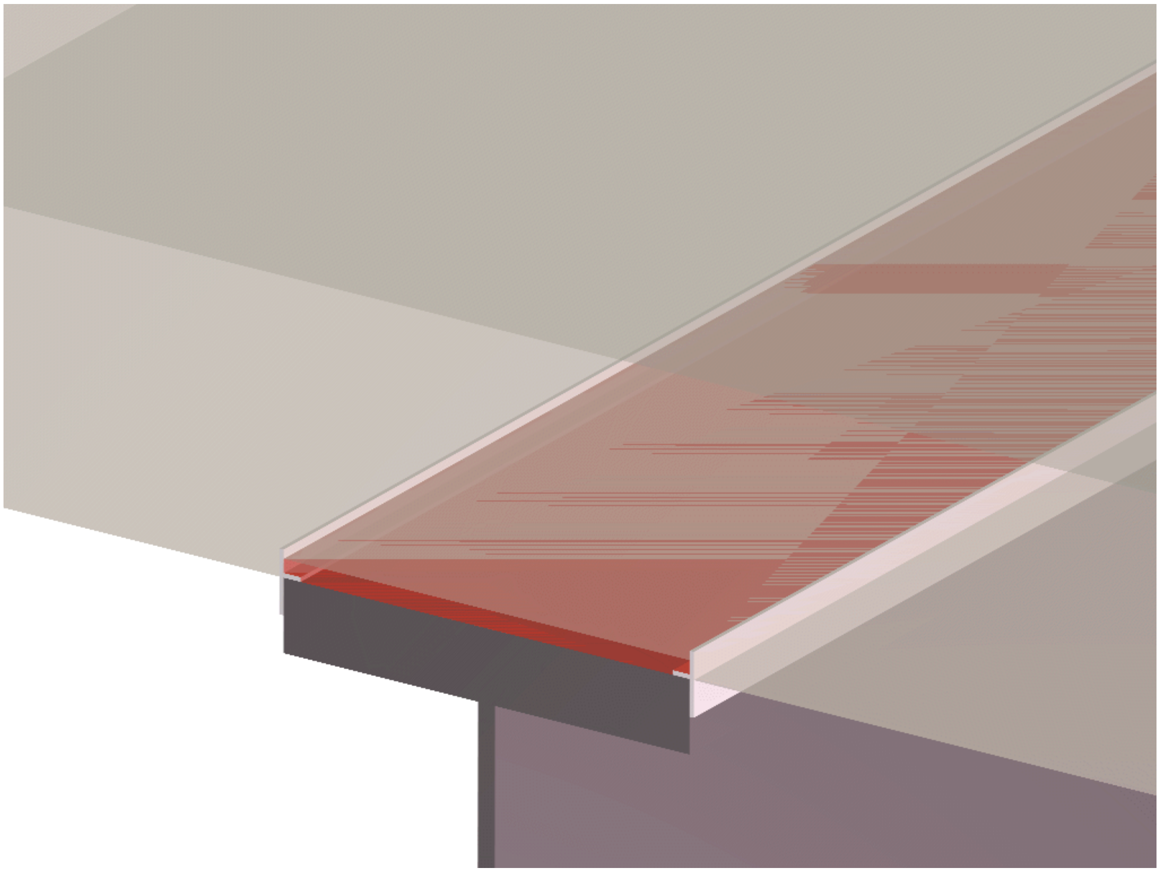

Figure 3-108: Close-up of Final System

The final system is shown in Figure 3-108. The layer of sand allows the epoxy to flow freely. The irregular interface created by the concrete exposes more surface area to the epoxy creating a stronger bond. Additionally, the layer of sand prevents the concrete from bonding to the steel surface. It is believed, based on the component testing, that one reason for the extremely low adhesion of the epoxy to the steel after pumping is the cement residue left on the steel after de-bonding has occurred.

A modification has also been suggested expands on the layer of sand which utilizing larger aggregate. A layer of large pebbles is glued to the top of the flange in the shop prior to shipment. An example of this can be seen in Figures 3-109 and 3-110.

Figure 3-109: Pebbles bonded to steel

Figure 3-110: Close-Up of Pebbles

From this point, the alternative would proceed similar to the previous utilizing a layer of sand followed by pumped epoxy after the deck had cured and shrunk. The irregular surface presented by the pebbles would increase the surface area for the epoxy to adhere to and also provide a degree of mechanical interlock.

This method would allow the use of an epoxy ideal for adhesion to steel surfaces for the pebbles while using the low viscosity epoxy for the pumping. Since the pebbles would be applied in a controlled environment, the connection would be of high quality. Use of the pebbles would also eliminate the shear stud tripping hazard.

3.7 NON-DELAYED

The current practice making the steel beam or girder act compositely with concrete is to use welded shear studs on the top flanges of the beam or plate girders. A typical construction sequence involves welding the shear studs in the shop, transporting the beams or plate girders to the field and placing them on the supports. Workers are then required to walk on the top surface of the beam or plate girders for placing the formwork between the adjacent beams or plate girders. This formwork acts as a temporary support for the wet concrete that is later poured on the beam or plate girder.

Before placing the formwork between the beams or plate girders and while the workers have to walk over the top of the beam or girder, a safety problem exists. Workers could trip over the shear studs. This is the main reason that OSHA has mandated that shear studs must be welded in the field beginning January 18, 2002. The problems with field welding shear studs are first, it is expensive, and second, it reduces quality. Field welding, in general, results in lower quality and the industry is very hesitant to do any welding in the field. The preference is to do all welding in the shop before the beams or girders are shipped to the field.

The new composite system is safer than using shear studs although it does not eliminate the tripping hazard altogether.

Figure 3-111 shows the proposed system. A continuous plate is welded to the top flange of the beam or plate girder. Preliminary engineering indicates that the dimensions of the plate could be on the order of 2 to 4 inches in height and 3/8 to 3/4 inches in thickness. Welding to attach the plate to the top flange of the beam or plate girder could possibly be on one side or both sides of the plate. Welds could possibly be continuous or discontinuous. The welding of the plate to the beam or plate girder is accomplished in the shop prior to shipment to the field.

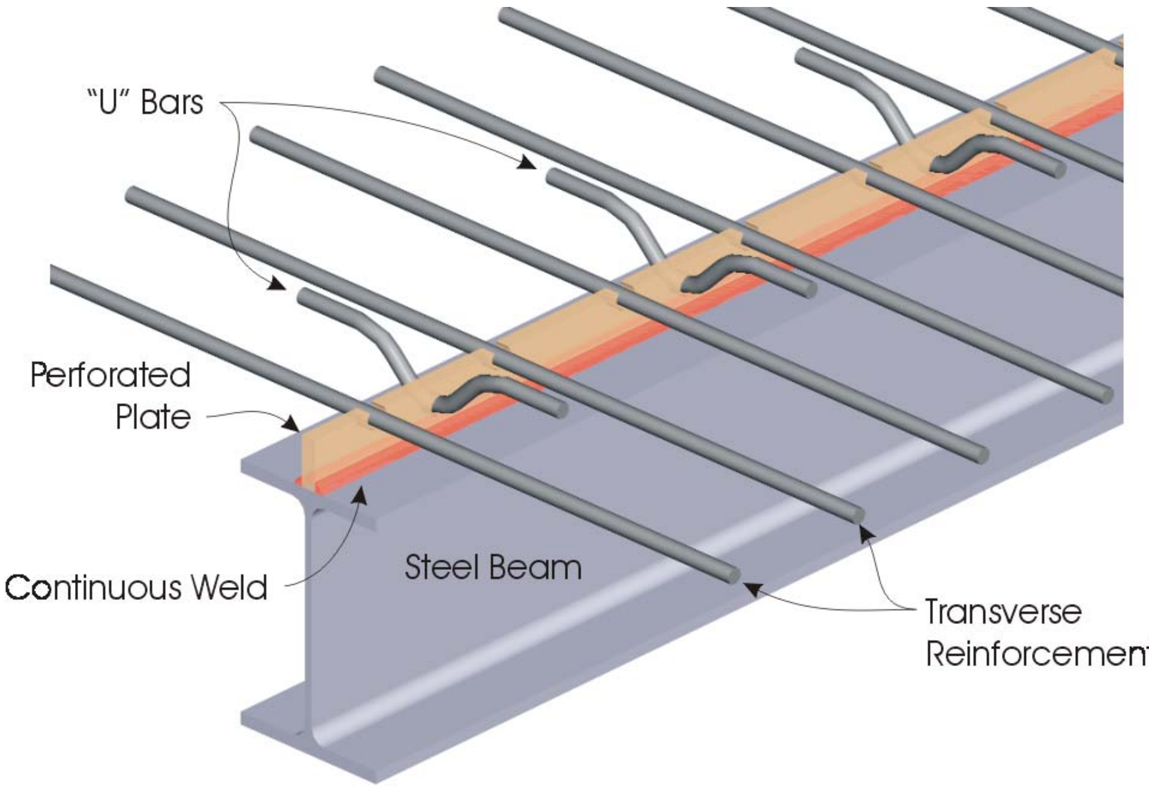

Figure 3-111: Illustration of Drop Bar Alternative

The plate shown in Figure 3-111 has complete holes placed at intervals. These complete holes are for passing a U shaped drop reinforcing bars through. Figure 3-111 shows drop shaped bars passed through the complete hole in the continuous plate. The U shaped bar assists in providing the composite action between the steel beam or plate girder and concrete. Additional holes can be placed, as shown in Figure 3-111, to accommodate the placement of transverse reinforcement in the slab.

The holes accommodating the transverse reinforcement would be in the form of an incomplete circle, as shown in Figure 3-111. This shape, after placement of the concrete, allows development of additional composite action between steel beams or plate girders and the concrete slab. The open top allows the transverse reinforcement to simple be placed through the top rather than inserted from the side. These holes also act as form chairs for the transverse reinforcement.

The drop bars are placed after the workers place the formwork. The drop bars are then tied to other reinforcing bars before casting the concrete.

3.7.1 Experimental Testing of New Composite Action System

To develop an approximate idea on the potential of the new system, a very small scale test consisting of a composite steel beam utilizing the proposed system was constructed and an ultimate load test was carried out. This section of the report provides results of this test.

3.7.2 Design of the Specimen Tested

The objective was to design and test a very simple specimen and evaluate "qualitatively" the merits of the new system. The design criteria were identical to the epoxy beam prototype tests performed earlier. In fact, since the slab in the epoxy beam test separated at such a low load, it was determined that the stress in the steel joist remained below 50% of yield. Therefore, the same steel from the epoxy beam test was used for the current test. However, the dimensions of the concrete slab were modified slightly.

The selected specimen consisted of a 10 foot long W8×21 Grade 50 steel beam topped by a 30 inch wide by 6 inch thick concrete slab with specified concrete compressive strength of 7500 psi. A 3/8"×5-1/2" plate was welded to either side of the web to increase the shear capacity. Slab reinforcement consisted of a top and bottom layer of #4 bars (5 inches on center) in both the longitudinal and transverse directions. The transverse bars were positioned closest to the slab face with one inch of cover both top and bottom. Figure 3-112 illustrates the cross section of the test specimen.

The predicted moment at which the bottom flange would begin to yield, assuming fully elastic behavior, was determined to be 3,020 kip-in while the ultimate load predicted from a moment curvature analysis using assumed nonlinear material properties was 4,600 kip-in. The shear flow required at ultimate load would be 9.4 kips/in.

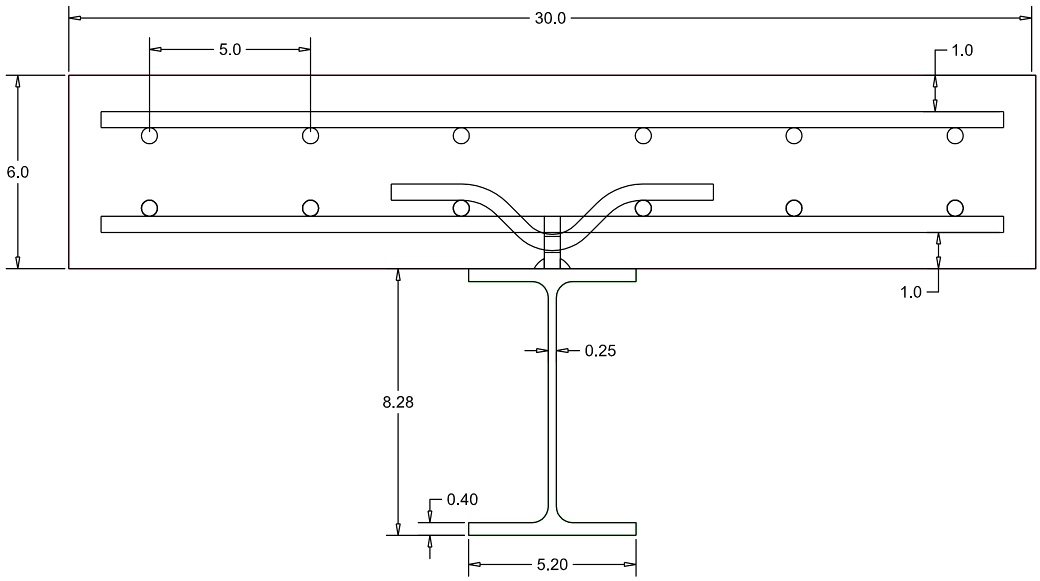

Figure 3-112: Test Specimen Dimensions

3.7.3 Design of Shear Transfer Mechanism for Specimen Tested

Figures 3-113 and 3-114 show details of the plate and U bars used for the test specimen. Figure 3-113 shows the holes for allowing the passing of the bottom layer of the transverse reinforcement and the drop bars. The plate used was a 1/4"×1-5/8" longitudinally welded to the middle of the top flange. Two styles of perforations were plasma cut into the plate. The first style created a saddle for the transverse rebar by cutting a circular hole which was cropped by the top of the plate. These were placed 5 inches on center, the same as the transverse bar spacing. The second style of perforation was through cut holes placed approximately at mid depth in the plate. Number 4 drop bars were placed in these holes. The drop bars were tied to longitudinal bars in the slab before casting the concrete. The drop bars were placed 10 inches on centers and set between the transverse bars. The plate was welded to the top flange with staggered 5/16" fillet welds 4-

1/2" long on 6" centers on alternating sides of the plate. The staggered weld could be replaced by a continuous weld to facilitate automation.

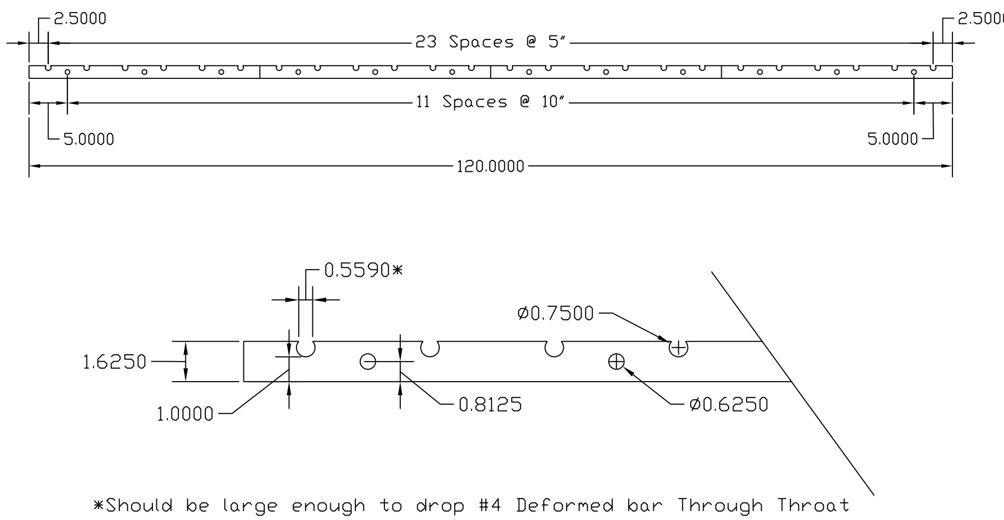

Figure 3-113: Perforated Plate

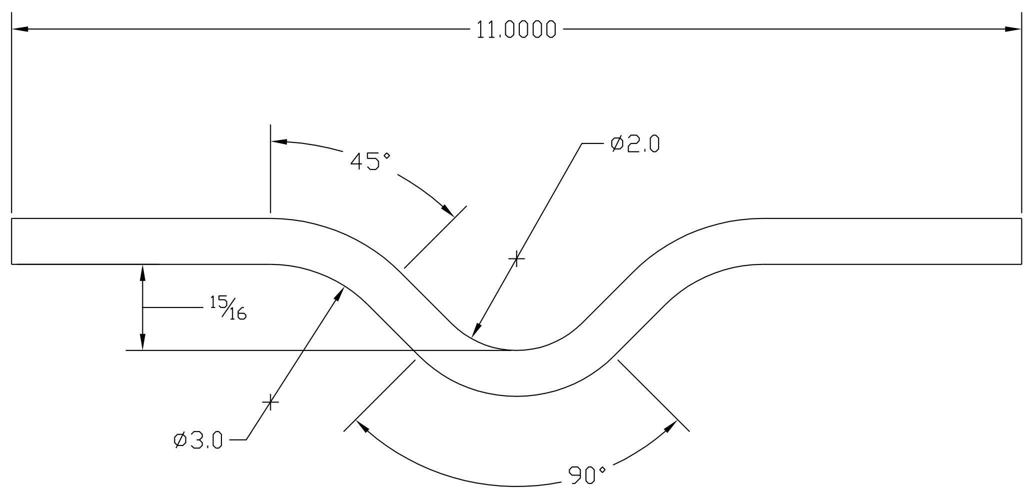

Figure 3-114: Drop Bar

3.7.4 Testing

Figure 3-115 shows the test specimen and test setup before start of the test. The specimen was supported on rollers at the ends of a 100 inch span. One end was free to move longitudinally. The specimen was tested by applying a point load under deflection control at midspan. The load was applied in evenly spaced increments and data was taken at each stage.

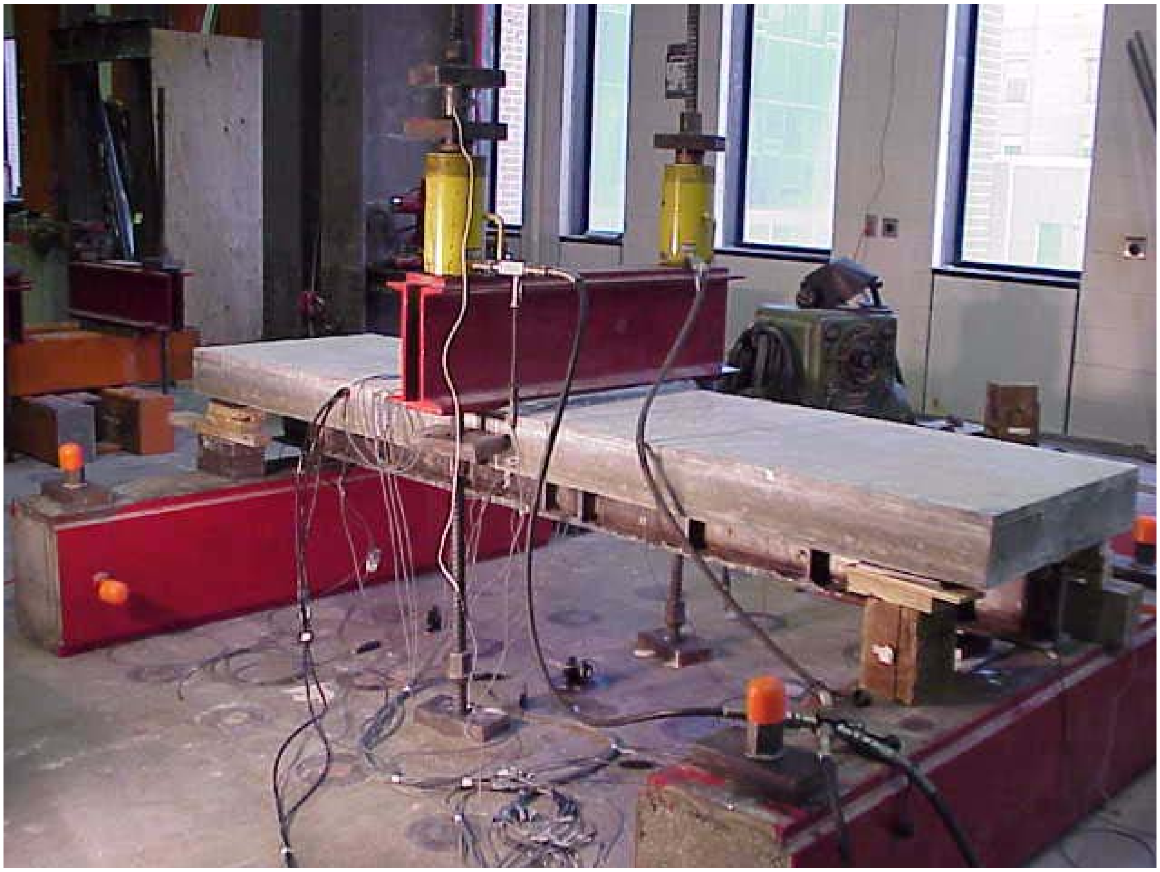

Figure 3-115: Test Setup

Figure 3-116 shows the load deflection plot obtained from the test. Also shown in the figure is the predicted yield load and predicted ultimate load.

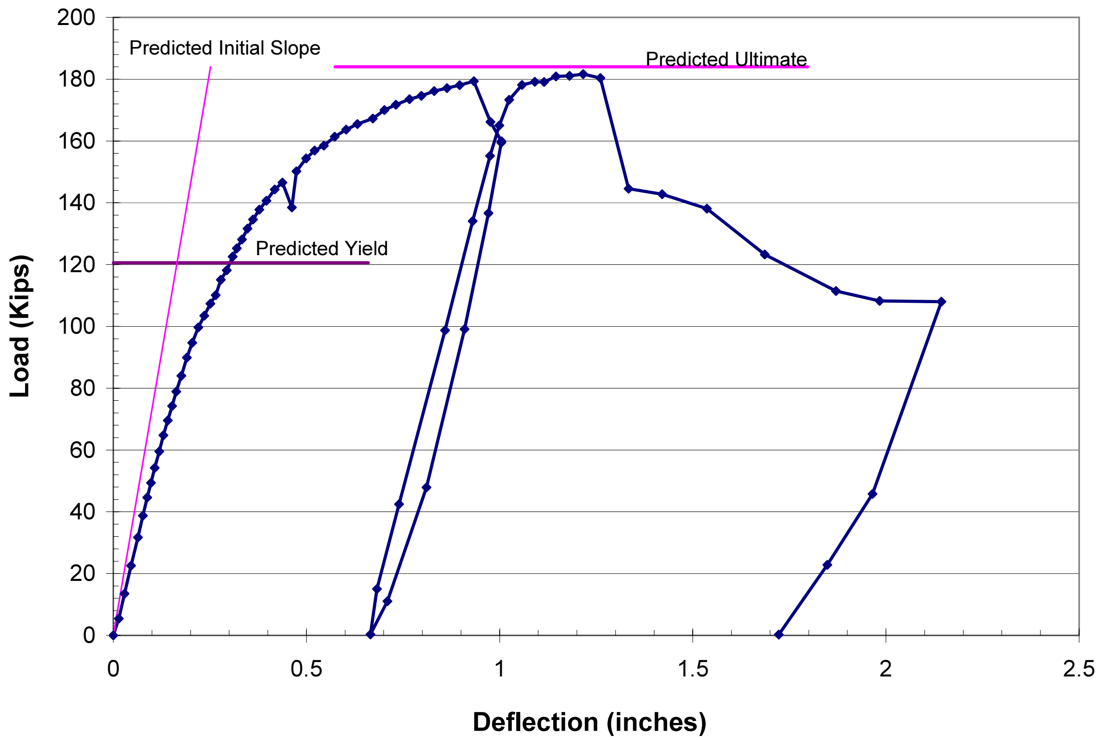

Figure 3-116: Load Deflection Curve

Figure 3-117 shows a photo of the test specimen after the test conclusion. The specimen failed by crushing of the concrete slab, as indicated in Figure 3-1177. There was some relative slip between the slab and the steel as shown in Figure 3-118. Figure 3-119 shows the photo of the specimen after removal of the damaged concrete. There was no damage to the longitudinal plate or drop bars. The holes that were used to pass the transverse reinforcement and drop bars did show signs of inelastic deformations as shown in Figure 3-120. This indicates that, in designing the longitudinal plate, the bearing capacity of the holes should be a design item.

The results of the experiment were very encouraging. As seen in Figure 3-116, the specimen nearly reached the ultimate load as predicted by the moment curvature analysis. Further, the system sustained a large amount of inelastic deformation prior to failure.

Figure 3-117: Specimen after Failure

Figure 3-118: Relative Slip of Concrete and Steel at End of Girder

Figure 3-119: Specimen with Concrete Removed

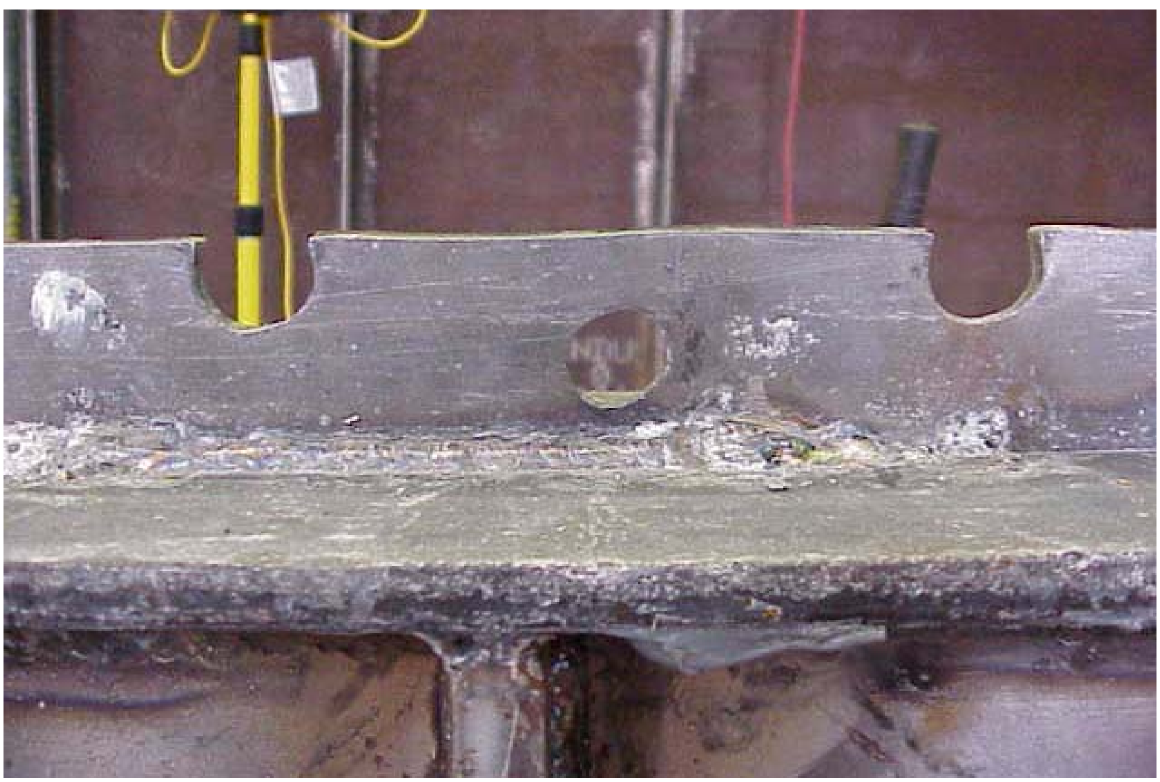

Figure 3-120: Close up of Perforated Plate after Test

Chapter

Summary and Conclusions 4

Cracking of concrete decks is a costly maintenance item. The transverse cracks that form before opening the bridge to traffic allow moisture to penetrate the deck and are responsible, in part, for corrosion and deterioration of the deck concrete. It is well accepted that one of the primary reasons for development of transverse cracks in bridges is the restraint that is provided by such elements as shear studs. Another reason for attempting to eliminate shear studs is safety. The shear studs pose a serious tripping hazard to workers who must walk on the girders, especially prior to the placement of formwork.

Therefore, developing a bridge system where composite action is developed after the concrete is hardened will reduce greatly the extent of transverse cracking observed in bridge decks and elimination of shear studs in particular would reduce the potential tripping hazard to workers.

To this end, research was conducted at the University of Nebraska - Lincoln as a pilot study to identify potential alternatives which would address these safety and transverse cracking problems. This research included a number of component level tests along with two prototype beam tests. The objective of the research was to recommend an alternative to the Nebraska Department of Roads Bridge Division for further evaluation.

4.1 RECOMMENDATIONS

Of the alternatives considered, two appear to offer the most promise for eventual implementation. They are the epoxy grouted studs and the drop bar alternatives. In addition to the specific alternatives which are recommended, there are also several general recommendations which arise as a result of the experimental investigations.

4.1.1 GENERAL

Based on the results of the investigations, any system which relied on epoxy to transfer the shear force would also require at least intermittent shear studs or other mechanical connectors to transfer the vertical component of shear.

Further, it appears as though the systems constructed using epoxy fail in a very sudden non-ductile manner. One suggested remedy would be intermittent shear studs at large spacing to provide some ductility. However, due to compatibility considerations, it is believed the two mechanisms would not act in parallel, but rather series. Since the epoxy connection would be very stiff and not elongate, the shear studs would not assist in carrying the load until after the epoxy had failed. At this point, the only transfer mechanism would be the remaining shear studs. Since the number of shear studs had been deliberately chosen such that they were not able to carry the entire load, they too would fail resulting in a complete loss of composite action.

4.1.2 Epoxy embedded studs

Although the epoxy grouted stud alternative was not investigated experimentally, its method of shear transfer is identical to that of conventional construction which leads one to believe that the behavior of the grouted studs would be similar to the conventional construction.

There are, of course, a few details still in contention which would require further research. One of the most basic elements needed for implementation of the concept is the boots themselves. Many of the factors relating to the final design of the boots would relate to plastics manufacturing and would require input from that field. However, some other factors, such as the ideal taper and dimensions would depend on structural considerations.

Removal of the boot could be difficult. If it were determined that the boot could remain and simply be filled with grout, this could further simplify the system. One potential argument for not leaving the boot in would be that moisture could seep around the outside of the boot and reach the steel. This could possibly be prevented by corrugating the outside of the boot or utilizing some other texturing to assure a barrier is formed.

4.1.3 Drop bar alternative

A prototype beam utilizing the drop bar alternative was tested and the results were quite positive. The behavior of the test specimen was nearly identical to the behavior assuming complete composite interaction as determined using moment curvature analysis.

The primary advantage of the drop bar system is the elimination of the tripping hazard posed by shear studs. The system utilizes a single plate welded along the length of the girder protruding vertically in the middle of the flange. This allows workers to place a foot on either side while walking along the girder. Similar systems have been investigated in the past (Oguejiofor and Hosain 1992; Medberry and Shahrooz 2002). However, previous research required that the transverse bars be threaded through holes in the plate which would be quite difficult in bridge construction where the transverse bars are as long as the full width of the deck. In the current system, semicircular notches placed near the top of the plate allow the transverse bars to be dropped into position rather than passed through the plate. Intermittent full through holes through which is passed a specially shaped drop bar provides for transfer of the vertical component of shear.

A number of aspects of this system would require further investigation before a final decision could be made regarding the system's potential for implementation.

4.2 Summary of rejected alternatives

In the course of the research a number of alternatives were envisioned. As with any study of this nature, many alternatives were quickly dismissed. A number of alternatives were given some more consideration, however not recommended. The following sections summarize those alternatives which were not recommended and their advantages and disadvantages.

4.2.1 Epoxy Injection

The first option explored using epoxy to literally glue the concrete deck to the supporting girders. At the time of deck casting, a hollow tube is embedded vertically in the concrete. This tube provides a conduit for the epoxy which is pumped under pressure after the concrete deck has shrunk during curing.

Advantages

- Minimal modification to existing construction methods
- Eliminates Tripping hazard of shear studs
- Inexpensive
    - Low cost equipment
    - Small volume of epoxy

DISADVANTAGES

- Low shear strength
- Require additional mechanism to resist vertical component of shear
- Non-Ductile Failure

4.2.2 MECHANICAL ALTERNATIVES

Mechanical alternatives utilize a device embedded in the concrete however not attached to the steel girder. After the concrete has cured and the desired shrinking has been allowed to take place, the device is then connected to the steel girder. Details utilizing bolts or threaded studs are available.

ADVANTAGES

- Strong and ductile
- Installation is Flexible
- Eliminates Tripping hazard of shear studs

DISADVANTAGES

- Expensive
  Initial Cost
  Continued Inspection and Maintenance
- Connection provides incipiency to corrosion

4.2.3 PRE-CAST OPTION

A second epoxy alternative used a much more viscous epoxy than that used for the injection alternative. As a result, the epoxy would need to be applied directly to the steel girder itself. This could be done if the deck were composed of pre-cast sections.

ADVANTAGES

- Makes use of higher strength epoxy
- Pre-Cast Deck
  This can also be a disadvantage depending on circumstances

Disadvantages

- Pre-Cast Deck
- Requires additional mechanism to resist vertical component of shear
- Non-Ductile Failure

Combination Methods

Several alternatives were developed which attempt to combine advantages of the epoxy and mechanical systems.

4.2.4 Stud Strip

Shear studs are set in a strip of low modulus material laid on the top of the steel girder. When the deck is cast, the studs are embedded in the concrete. After the deck has cured, epoxy is pumped between the strip and the steel girder.

Advantages

- Makes use of higher strength epoxy
- Modular system
- Eliminates Tripping hazard of shear studs
- Utilizes Mechanical interlock to concrete

Disadvantages

- Cost
  Fabrication of Stud Strip
  Research and Development
- Concept has not been tested

4.2.5 Mixed aggregate method

Several alternatives were examined which used a mix of aggregates to improve the performance. The first of these simply used a layer of sand covering the top flange prior to placing the concrete deck

Advantages

- Similar to pumped epoxy alternative

- Increased adhesion to concrete

DISADVANTAGES

- Similar to pumped epoxy alternative
- Untested

A second alternative utilizing aggregate is to glue on a layer of large pebbles in the shop prior to shipment. From this point, the alternative would proceed similar to the previous utilizing a layer of sand followed by pumped epoxy after the deck had cured and shrunk.

ADVANTAGES

- Similar to pumped epoxy alternative
- Increased adhesion to concrete
- Utilizes high performance epoxy in connection to steel
- Some degree of mechanical interlock

DISADVANTAGES

- Similar to pumped epoxy alternative
- Untested

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