FHWA/IN/JTRP-2007/8

Final Report

**DECK REPLACEMENT – USE OF EXTRA
COATING THICKNESS EPOXY-COATED
BARS**

***(Innovative Bridge Construction Program
Project)***

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(Innovative Bridge Construction Program Project)

Introduction

The thickness of the ASTM A 775 epoxy-coating on the reinforcing bars used in normal-weight concrete bridge decks currently is to be between 8 and 13 mils (INDOT 2008). These values are very similar to the current AASHTO LRFD (2004) values of 8 to 11 mils of flexible coating. The results of this study were implemented in the form of design recommendations for bond of epoxy-coated bars with thicker coatings up to 18 mils. This recommendation is consistent with the range proposed in the study “*Methods of Corrosion*

Protection and Durability of Concrete Bridge Decks Reinforced with Epoxy-Coated Bars – Phase I.” The scope of the work included evaluation of AASHTO LRFD (2004) and ACI 318 design recommendations for the development and splice length of extra epoxy-coating thickness reinforcing bars in normal-weight concrete bridge decks and a proposed monitoring plan for a generic concrete bridge deck in Indiana where epoxy-coated bars with thicker coatings might be employed.

Findings

The focus of the study was the evaluation of the performance in bond to normal-weight concrete of epoxy-coated bars with coating thickness up to 21 mils using No. 5 and No. 9 bars. Single splices as well as splices in bundled bars were evaluated. The results of the experimental program, consisting of 20 splice specimens tests and an extensive literature review of relevant works, yielded the following findings:

- (i) For each bar size specimen designed for $1/2 f_y$, the stress at failure obtained using beam theory analysis at the Failure Load, were similar when comparing the Bundled Bar with the Single Splice specimens.
- (ii) Increasing the coating thickness up to 21 mils resulted in ratios of test to calculated values at failure greater than 1.0 when the calculated value was determined using the AASHTO LRFD (2004) Specifications and the ACI 318-05 Code. ACI 318-05 calculated values resulted in an average test/calculated ratio slightly lower than the average for test/calculated ratios using the AASHTO LRFD (2004) Specifications. However, the ratio of test/calculated stress for the No. 9 bundled bar were greater for ACI than for AASHTO.
- (iii) The specimens containing bars with coatings thickness in the range of 18-21 mils did not show significantly different deflections at failure when compared to those of the companion specimens with bars having a coating thickness range of 12-15 mils.
- (iv) For the No. 9 Bundled Bar specimens designed for $1/2 f_y$, the increase in coating thickness resulted in a

- reduction** in the average crack spacing and average crack width with thicker coatings.
- (v) For the No. 9 single bar splices the increase in coating thickness resulted in a reduction in the average crack width. The reduction was less significant than that observed in the No. 9 Bundled Bar specimens. Furthermore, the increase in coating thickness resulted in no change in the average crack spacing.
 - (vi) For the No. 5 Single Splice specimens designed for $1/2 f_y$, the increase in coating thickness resulted in an **increase** of average crack spacing and average crack width for thicker coating thicknesses.
 - (vii) Bundling of bars resulted in an increase of the average crack spacing compared to that observed in Single Splice specimens only in the case of the No. 9 bar specimens with 12-15 mils coating thickness. In the other three cases considered, No. 5 bar specimens with 12-15 mils coating thickness, No. 5 bar specimens with 18-21 mils coating thickness and No. 9 bar specimens with 18-21 mils coating thickness, bundling of bars resulted in a decrease in the average crack spacing when compared to similar Single Splice specimens. The largest reduction was observed in the No. 5 bar specimens with coating thickness in the range of 18-21 mils.

Implementation

The use of the current provisions for development and splice length of epoxy-coated bars in tension in both the AASHTO LRFD (2004) and ACI 318-05 is supported by the test findings of the experimental program up to a coating thickness not to exceed 21 mils. However, since the ACI 318-05 specifications consider the critical parameters of cover and

transverse reinforcement, the authors encourage the Indiana Department of Transportation to use these provisions in the design of development and splice length of bars with coating thickness up to 18 mils. INDOT 700 Committee is implementing the results of this study through a change in the specification for epoxy-coated bar thickness.

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Epoxy-Coated Bars**

(Innovative Bridge Construction Program Project)

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Indiana Department of Transportation or the Federal Highway Administration at the time of publication. This report does not constitute a standard, specification, or regulation.

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1 INTRODUCTION AND MOTIVATION

1.1 Introduction

In this study, the bond strength of epoxy-coated bars with thicker coatings, 12 to 21 mils, is evaluated. The thickness of the ASTM A 775 epoxy-coating on the reinforcing bars was specified to be between 6 to 12 mils in INDOT's 1999 Standard Specifications, and currently, the epoxy thickness is to be between 8 and 13 mils (INDOT 2008). These values are very similar to the current AASHTO LRFD (2004) values of 8 to 11 mils of flexible coating.

The hazard associated with corrosion of steel reinforcement in Indiana has been mitigated through a combination of good quality concrete, adequate concrete cover and the use of epoxy-coated bars. The standard practice in new concrete bridge decks has been the use epoxy-coated steel in conjunction with a minimum top concrete cover of 2.5 inches.

In the study entitled "*Methods of Corrosion Protection and Durability of Concrete Bridge Decks Reinforced with Epoxy-Coated Bars – Phase I*" for the Indiana Department of Transportation through the Joint Transportation Research Program (Samples and Ramirez 1999), a total of a 123 bridge decks were surveyed, including eleven concrete bridge decks under construction. In particular, the field evaluation of the eleven concrete decks under construction indicated that increasing the flexible coating thickness required in epoxy-coated reinforcement would dramatically decrease the damage created to the bars during the bridge deck casting operation. It was also determined that an increase on average of 4 mils in the thickness of the epoxy-coating reduced the number of defects incurred during the concrete casting operation when using the pump method by an average of 73%. Lowering the pump to reduce the vertical drop of the concrete also reduced the number of holidays created by an average of 50%. Maintaining the integrity of the coating is essential for the effectiveness of epoxy-coated bars as a viable element of the corrosion protection system adopted in Indiana.

The evaluation of the existing decks conducted in the same study indicated that epoxy-coated reinforcement combined with Class C concrete provided the most successful corrosion protection method as only 11% of the bridge decks inspected in this category during the initial bridge inspections showed signs of corrosion distress. On the other hand, uncoated reinforcement and a design cover of 1.0 in. of Class C concrete and 1.5 in. of latex modified overlay was not an effective corrosion protection method as 52% of the bridge decks inspected in this category during the initial bridge inspections showed signs of corrosion distress. It was also shown that cracking and insufficient concrete cover may decrease the effectiveness of epoxy-coated reinforcement as a corrosion protection system. Corrosion of the epoxy-coated reinforcement was discovered during the detailed bridge inspection in areas of cracking and shallow cover.

In the same study an extensive laboratory phase to evaluate the corrosion performance of epoxy-coated bars was conducted. The results of the laboratory phase indicated that corrosion of epoxy-coated reinforcement can be prevented with a high mat to mat

resistance. A high mat to mat resistance can be provided by the use of epoxy-coated reinforcement with limited damage to the coating. Both the field and the laboratory evaluations showed that a thicker epoxy-coating will limit the amount of damage to the coating, which will increase the mat to mat resistance when utilizing epoxy-coated reinforcement. It was recommended that an increase of 6 mils to the minimum coating thickness of steel reinforcing bars be implemented. This increase implies an allowable range of 12 to 18 mils. It is expected that the use of a thicker coating will significantly decrease the damage to the epoxy-coating, and thus increase the effectiveness of epoxy-coated reinforcement as a corrosion protection system. The study also recommended an evaluation of the bond performance of bars with thicker epoxy-coatings.

1.2 Objective and Scope

This study extended the knowledge gained in the previous studies on the bond strength of epoxy-coated bars in normal-weight concrete. The results of the study were implemented in the form of design recommendations for bond of epoxy-coated bars with thicker coatings up to 18 mils. This recommendation is consistent with the range proposed in the study *“Methods of Corrosion Protection and Durability of Concrete Bridge Decks Reinforced with Epoxy-Coated Bars – Phase I.”* The scope of the work included evaluation of AASHTO LRFD (2004) and ACI 318 design recommendations for the development and splice length of extra epoxy-coating thickness reinforcing bars in normal-weight concrete bridge decks and a proposed monitoring plan for a generic concrete bridge deck in Indiana where epoxy-coated bars with thicker coatings might be employed.

1.3 Report Organization

In Chapter 2 of the report, a literature review is presented on the bond strength in tension of epoxy-coated mild deformed bars. Chapter 3 describes the experimental program conducted to evaluate the bond strength of reinforcing bars with extra coating thickness. The program was developed taking into account the results of the literature review of relevant works. Chapter 4 is a presentation of measured test data from the experimental program described in Chapter 3. Chapter 5 discusses the analysis of the data presented in Chapter 4. Finally, Chapter 6 contains the summary and conclusions of the study, proposed designed recommendations, and a proposed instrumentation plan for a future bridge deck in Indiana built with extra coating thickness bars. The references listed in Chapter 7 represent all the works addressed in this report.

The page numbers are at the bottom of each page; the first number is the chapter number and then the page number of the chapter, which is separated with a dash. The page numbers start over at the beginning of each new chapter. The tables and figures discussed in each chapter are presented at the end of that chapter. All the tables are presented before all the figures for a given chapter. The numbering notation for both the tables and figures is similar to the page numbers where a period separates the chapter

number and the table or figure number, with the label Table or Figure before the numbering notation.

2 LITERATURE REVIEW ON THE BOND STRENGTH OF EPOXY-COATED REINFORCEMENT

2.1 Introduction

The effect of the extra thickness of coating on the bond strength and performance of normal-weight concrete beams under static and repeated loading was studied at Purdue as part of the project “*Performance-Related Specifications for Concrete Bridge Superstructures.*” The results of the bond study are contained in Volume 4 of the project’s final report entitled, “*Bond of Epoxy-Coated Bars with Thicker Coatings*” by Appelhans and Ramirez (2002). The specimens tested by Appelhans represented typical construction practices in Indiana with regard to concrete strength, minimum cover and bar sizes (No. 5 bars and No. 8 bars). The specimens were designed to fail in splitting mode prior to yielding of the reinforcement to allow for a direct comparison with companion specimens reinforced with uncoated bars. The study showed that the relative bond strength between coated and uncoated bars remained the same for bars with coating thicknesses up to 18 mils.

A more recent study conducted by Miller et al. (2003) using beam end specimens (ASTM A 944), concluded that the increase in coating thickness with ASTM A 775 epoxy-coatings reduces the bond strength of smaller diameter bars (No. 5 and smaller) while larger diameter bars (No. 6 and larger) seemed to be almost not affected. This observation was based on the tests of specimens containing bars with flexible coating thicknesses in the range of 6.4 to 16.5 mils. It was noted that if the upper limit of coating thickness was increased to 20 mils, the bond strength of No. 6 deformed bars was reduced. The authors of the study concluded that the maximum allowable coating thickness could be increased from 12 mils to 16.5 mils for No. 6 and larger bars meeting the requirements of ASTM A 755M.

Additional considerations such as increasing the thickness of the coating to 21 mils, casting position, bundling of bars, transverse reinforcement and structural performance of code length splices remain to be investigated. The proposed research will add to the information provided by the two previously mentioned studies by addressing the performance of coated bars with specified epoxy-coating thickness in the ranges of 12 to 15 mils and 18 to 21 mils in reference to:

- Bar bundling, and
- Individual splice/development length.

Because this study specifically addresses the use of these bars in bridge decks, the role of transverse reinforcement and casting position were not selected as variables in the study. Bar bundling was selected from the standpoint of studying effects due to reduced perimeter of bar in contact with concrete.

In this study, the effects of increasing the allowable coating thickness of ASTM A 775 epoxy-coatings to a thickness range of 12 to 21 mils on the bond strength of deformed bars will be investigated. The findings of the study will be used to verify and, if

necessary, modify the current design specifications for bond strength to accommodate the use of epoxy-coated bars with thicknesses in the range of 6 to 18 mils. The verified/developed specifications will be used in the design of development and splice lengths of the mild reinforcement in the proposed deck replacement.

2.2 Background

Performance of reinforced concrete structures is closely connected to the bond between the concrete and the steel reinforcement. The steel reinforcement provides tensile strength to the concrete, which is strong in compression but weak in tension. Extensive research has been performed on the subject of bond between concrete and steel reinforcement for well over 100 years (Hyatt 1877). As reinforcement has evolved through the years from square deformed steel bars to circular deformed steel bars to coated steel circular deformed bars to fiber reinforced polymer (FRP) reinforcing rods, the research conducted has provided an ever-improving understanding of this aspect of reinforced concrete behavior.

In reinforced concrete structures exposed to harsh corrosive environments, epoxy-coating has been used to protect the reinforcement against corrosion. It is important to avoid damage to the coating to prevent corrosion. Coating can be damaged at various instances during construction. A possible strategy to minimize damage to the coating is to increase its thickness. It has been shown that the thicker the epoxy-coating, the more resistant this flexible coating is to imperfections during the construction process, and therefore affords better protection to the steel from corrosion (Samples and Ramirez 2000a,b). This research report focuses on the bond behavior and design of straight epoxy-coated steel deformed bars that are placed in tension while embedded in normal-weight concrete. Of particular interest is the bond performance of such bars as the thickness of the flexible coating is increased under monotonic loading conditions addressed in Chapter 12 of ACI 318. Studies have shown that epoxy-coating reduces the bond strength of steel deformed bars. Dynamic, blast and seismic loading are outside the scope of this research project.

ACI Committee 408, Bond and Development of Reinforcement, issued its first report on the subject in 1966. The report emphasized the importance of splitting cracks in governing bond strength. The pioneering concepts on the bond strength of reinforcing bars in concrete represented it in terms of the shear stress at the interface between the reinforcing bar and the concrete, effectively treating it as a material property. Over the years, the additional research has shown that bond strength is a structural property, dependent not only on the materials, but also on the geometry of the reinforcing bar and the structural member itself. It is also important to note that the knowledge base on bond is strongly rooted on empirical observations. An understanding of the observed experimental behavior is no doubt critical to the development of physical models and design techniques.

In the context of this report, a few words are appropriate with respect to terminology. The term bond force represents the force that tends to move a reinforcing bar parallel to its length with respect to the surrounding concrete. Bond strength represents the maximum bond force that may be sustained by a bar embedded in concrete. The terms

anchored length, bond length and embedded length are used interchangeably to represent the length of a bar over which the bond force acts. Bond stress is defined as the force per unit of bar length embedded acting around the perimeter of the bar.

2.2.1 Identification of Key Parameters

There are primarily two bond failure modes associated to the failure of the concrete surrounding the reinforcing bar: splitting and pullout. Splitting failure occurs when the thickness of cover of the confining concrete is not enough to resist the radial stresses before the maximum value of the bond stress (τ_{bu}) is reached (Figure 2.1). Figure 2.1 shows a theoretical representation of bond stress vs. slip (τ_b -s) for a deformed reinforcing bar anchored in concrete. The vertical axis represents the force per unit area, defined as bond stress. This stress acts around the perimeter of the bar along the length of embedment. The maximum adhesion bond stress (τ_{b0}), maximum value of the bond stress (τ_{bu}), and residual friction force during pullout failure (τ_{br}), are illustrated in this figure. The horizontal axis represents slip of the reinforcement with respect to the surrounding concrete. Pullout failure occurs when the maximum value of the bond stress (τ_{bu}) is reached in well confined concrete which prevents splitting, and the concrete lugs (concrete between the steel deformations) shear off and slip with the deformed reinforcement relative to the surrounding concrete. Both modes of failure are illustrated in Figure 2.2.

The transfer of forces from reinforcing bar to surrounding concrete occurs for a deformed bar by at least 3 well known mechanisms: adhesion, friction, and bearing. First, chemical adhesion, due to the cement paste curing, prevents slipping of the reinforcement with respect to the surrounding concrete. Slip occurs when the tensile force in the system exceeds the adhesion force. Once adhesion is overcome, it is never regained, and friction and bearing forces determine the bond strength of the deformed bar. Clamping action of friction forces come about due to the surface roughness of the reinforcement and the radial forces from the concrete confinement. Bearing forces come from the mechanical action of the ribs of the deformed reinforcement. Friction and bearing forces can act at the same time as seen in Figure 2.3a. After adhesion has overcome, in the case of deformed bars, most of the force is transferred by bearing. Epoxy-coated bars have shown that friction between the concrete and the bar deformations (ribs) plays a significant role in force transfer when their bond strength is compared with that of uncoated bars.

The role of the epoxy-coating on the bond strength of a deformed bar can be illustrated using the previously discussed key components of the bond strength of a deformed bar in concrete. Adhesion is overcome first with uncoated and coated rebar, although this happens at different levels of force for same bar size and similar anchorage conditions. As the bar begins to slip with respect to the surrounding concrete, friction and bearing components are engaged. Adhesion and friction are decreased due to the smooth surface the coating creates for epoxy-coated bars compared to uncoated steel bars (Figure 2.4). In this figure, the end region of a beam after shear failure clearly shows the epoxy-coated reinforcement (green bars) clean and free of concrete, while the black bars (vertical shear

reinforcement) are shown with concrete adhered particularly around the rib area. Figure 2.5 shows, after a bond test conducted in the study reported herein, how smooth and glassy the concrete surface, which is cast against the epoxy-coated bar, and how clean the epoxy-coated bar is after a splice failure. The decrease in friction due to epoxy-coating has been illustrated through a simple test of a flat-plate specimen (Cairns and Abdullah 1994). The test set up is shown in Figure 2.6 and the results of the test are shown in Figure 2.7. In general, adhesion and friction are seen as negligible for epoxy-coated bars as illustrated in Figure 2.3b, therefore bearing on the ribs is the main component of the bond strength for epoxy-coated bars. With the same radial stress, the bond stresses are lower in the confining concrete with an epoxy-coated bar compared to an uncoated bar that has friction with the bearing component (Figure 2.3). As a consequence for the same bond force in the bar, the radial pressure (splitting stress) is also higher with the presents of epoxy-coating.

Bond strength is influenced by several factors that affect the three basic components of bond between concrete and steel. Adhesion and friction are greatly influenced by the strength of the cement paste and surface condition of the reinforcement, whether it is smooth or rough, or if rust or coating is present. Concrete compressive strength affects the bearing strength of the ribs on the concrete since the concrete will crush in compression at a certain bearing stress. Tension forces are induced in the surrounding concrete due to radial stresses and bearing stresses and cause cracking in the concrete, therefore the concrete tensile strength is an important factor in bond. The size of tension cracks without bond failure is influenced by the amount of concrete confining the reinforcement, which depends on the concrete cover and distance to other reinforcement in tension. Transverse reinforcement, if present, also controls these tension cracks by taking up some of the tensile stresses present in the concrete. The geometry of the ribs on the reinforcement, for instance the relative rib area (Figure 2.8), R_r ratio of bearing area of the ribs to the shearing area between the ribs, and the deformation pattern, also play a role on the bearing strength of the ribs.

2.2.2 Test Methods

In this section, an attempt will be made at quantifying the contribution of the parameters discussed in Section 2.2.1. This has been done in the past primarily through experimentation. Different types of test specimens have been used to study the relationship of bond between concrete and reinforcement as shown in Figure 2.9. The pullout specimen (Figure 2.9a) is easily fabricated and tested, but is the least realistic due to the transverse compression induced against the bar. This transverse compression increases the bond strength and therefore is not a realistic representation of bond for deformed bars in a structure. The beam-end, beam anchorage, and splice specimens do not show the transverse compression. The beam-end specimen (Figure 2.9b) is also easily fabricated, but tends to show slightly higher bond strength than the beam anchorage and splice specimens. The reason is the lack of additional cracks along the anchorage length where cracks would result in changes of the steel stress along the anchorage length. The splice specimen (Figure 2.9d) allows the formation of cracks in the constant moment region randomly and thus is the preferred method to evaluate the

strength of splices. The majority of the data used to determine the design equations for development and splice length of deformed bars in ACI 318 have been from splice specimens (Figure 2.9d). Even though the pullout and beam-end specimens are not the most realistic representation of bond in structures, they are effective for measuring the slip of the embedded bar at both the loaded end and the unloaded end. The loaded end slip is measured at the end of the specimen where the bar is loaded with the tension force, and the movement of the bar with respect to the concrete reflects the strain of the bar over the embedment length. The unloaded end slip, or free end slip, is measured at the opposite end of the specimen where there is no tension applied to the bar, and the bar does not start to slip until the whole embedment length is engaged. Slip is generally plotted against load, which is the total tension force applied to the bar.

2.2.3 Epoxy-Coating Effect

In U.S. practice, design specifications are written for bars with epoxy-coating thickness in the range of 7 to 12 mils (ASTM A 775M). The effect of epoxy-coating thickness on the load-slip curve is shown in Figure 2.10. In this figure, No. 5 bars, uncoated, epoxy-coated with 5 mils, and epoxy-coated with 12 mils were tested using beam-end specimens, with a bonded length of 3½ in. and clear cover of 1¼ in., or $2d_b$ (Choi, Hadje-Ghaffari, Darwin, and McCabe 1991). The slip is measured at the unloaded end, and splitting bond failure occurred in all tests. The epoxy-coating is a compressible material, and therefore in the initial load-slip curve a larger coating thickness results in greater slip. As the load is increased and bond failure is approached, the load-slip curves converge for both coating thicknesses. However, as the epoxy-coating thickness increases, the effective rib height can be decreased to a point that the rib is not as effective for bearing (Figure 2.11). Furthermore, there is a bar size effect with the rib height being proportional to the bar diameter, and therefore the effect is more pronounced in smaller diameter bars where the same epoxy-coating thickness covers more of the valley. Figure 2.12 shows this bar size effect comparing different size bars with different thicknesses of epoxy-coating (Choi, Hadje-Ghaffari, Darwin, and McCabe 1991). No. 5, No. 6, and No. 8 bars* were tested using beam-end specimens (Figure 2.9b), and their bonded lengths were 3½ in., 4½ in., and 8 in. respectively. The cover for all beam-end specimens was $2d_b$. The vertical axis represents the bond strength of coated to uncoated ratio (C/U). The horizontal axis is the epoxy-coating thickness measured with a pulloff-type thickness gage. C, N, and S represent different deformation patterns, and the bars used in this study have similar deformation patterns compared to the C and N bars. There is a greater effect of coating thickness on the smaller No. 5 bar than on the larger bars (No. 6 and No. 8). It can also be seen that the scatter of the data is made more pronounced by including the three deformation patterns.

Experimentation is crucial to quantify the factors that affect bond and to assist in the development of design specifications. Numerous experimental results have been published studying the effect of epoxy-coating on bond in reinforced concrete compared to uncoated reinforcement (Choi, et al 1991; Cleary and Ramirez 1991; Cleary and

* Relative rib area, R_r , was not reported for these bars.

Ramirez 1993; Darwin, Tholen, et al 1996a; Hasan, et al 1996; Hester et al 1993; and Treece and Jirsa 1989). However, only a few test programs have studied the effects of thickness of epoxy-coating on bond beyond the current upper limit in the U.S. specifications.

2.3 Code Design Procedures and Requirements

Research has a practical application and is reflected in design applications. Equations used for design purposes are empirically based on mainly splice specimen (Figure 2.9d) tests results. In this section, the current and proposed equations for development length for deformed bars in tension will be presented.

2.3.1 AASHTO LRFD (2004) Design Procedure

The tension development length (ℓ_d) for deformed bars in tension in AASHTO (2004, 5.11.2.1) is the product of the basic tension development length (ℓ_{db}) and the modification factor or factors. The current equations for the basic tension development length (ℓ_{db}) in in. are:

$$\text{For No. 11 bar and smaller} \quad \ell_{db} = \frac{1.25 A_b f_y}{\sqrt{f'_c}} \quad (2.1)$$

but not less than $0.4 d_b f_y$.

$$\text{For No. 14 bars} \quad \ell_{db} = \frac{2.70 f_y}{\sqrt{f'_c}} \quad (2.2)$$

$$\text{For No. 18 bars} \quad \ell_{db} = \frac{3.5 f_y}{\sqrt{f'_c}} \quad (2.3)$$

$$\text{For deformed wire} \quad \ell_{db} = \frac{0.95 d_b f_y}{\sqrt{f'_c}} \quad (2.4)$$

where:

- A_b = area of bar or wire, (in.²);
- f_y = specified yield strength of reinforcing bars (ksi);
- f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi); and
- d_b = the diameter of bar or wire (in.).

The modification factors that increase ℓ_d (AASHTO 5.11.2.1.2) are:

- For top horizontal or nearly horizontal reinforcement, so placed that more than 12.0 in. of fresh concrete is cast below the reinforcement, 1.4
- For lightweight aggregate concrete where f_{ct} (ksi) is specified, $\frac{0.22\sqrt{f'_c}}{f_{ct}} \geq 1.0$
- For all-lightweight concrete where f_{ct} is not specified, 1.3

- For sand-lightweight concrete where f_{ct} is not specified, 1.2

Linear interpolation may be used between all-lightweight can sand-lightweight provisions when partial sand replacement is used.

- For epoxy-coated bars with cover less than $3d_b$, 1.5
- For epoxy-coated bars not covered above, 1.2

The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy-coated bars need not be taken to be greater than, 1.7.

The modification factors that decrease ℓ_d (AASHTO 5.11.2.1.3) are:

- Reinforcement being developed in the length under consideration is spaced laterally not less than 6.0 in. center-to-center, with not less than 3.0 in. clear cover measured in the direction of the spacing, 0.8
- Anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis, $\frac{(A_s \text{ required})}{(A_s \text{ provided})}$
- Reinforcement is enclosed within a spiral composed of bars of not less than 0.25 in. in diameter and spaced at not more than a 4.0 in. pitch, 0.75

2.3.2 ACI 318-05 Design Procedure

The equation for development length for deformed bars in tension in ACI 318-05 (12-1) is:

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \lambda}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \quad (2.5)$$

in which the term $(c_b + K_{tr})/d_b$ shall not be taken greater than 2.5. This bond strength equation is based on Orangun, Jirsa, and Breen's (1975, 1977) equation to describe bond strength of bars with and without transverse reinforcement, but their original equation is multiplied by 90% and the 200 that was in the numerator is removed. The different parameters used in this section are defined as:

- A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and crosses the potential plane of splitting through the reinforcement being developed or lap spliced (in.²);
- c_b = smaller of (a) the distance from center of a bar to nearest concrete surface, and (b) one-half the center-to-center spacing of bars being developed (in.);
- d_b = nominal bar diameter of developed or lap spliced bar (in.);
- ℓ_d = development length in tension of deformed bar (in.);
- f'_c = specified compressive strength of concrete (psi);
- $\sqrt{f'_c}$ = square root of f'_c , expressed in psi units;

- f_y = specified yield strength of reinforcement (psi);
 f_{yt} = specified yield strength f_y of transverse reinforcement (psi);
 K_{tr} = $\frac{A_{tr} f_{yt}}{1500sn}$, transverse reinforcement index as defined in ACI 318-05 (12-2) (in.). It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present;
 n = number of bars being developed or lap spliced along plane of splitting;
 s = maximum center-to-center spacing of transverse reinforcement within ℓ_d (in.);
 λ = where lightweight concrete is use, **1.3**; where normalweight concrete is used, **1.0**;
 ψ_e = coating factor. for epoxy-coated bars with cover less than $3d_b$, or clear spacing less than $6d_b$, **1.5**; for all other epoxy-coated bars, **1.2**; for uncoated reinforcement, **1.0**;
 ψ_s = bar size factor. for No. 6 and smaller bars, **0.8**; for No. 7 and larger, **1.0**; and
 ψ_t = reinforcement location factor. where horizontal reinforcement is placed such that more than 12 in. of fresh concrete is cast below the development length or splice, **1.3**; for other situations, **1.0**;
 However, the product $\psi_t \psi_e$ need not be taken greater than **1.7**.

2.4 Summary

This literature review on the bond strength of epoxy-coated reinforcement conducted in this chapter provides the background for the selection of specimens in the Experimental Program, which is presented in Chapter 3. The current AASHTO specifications do not reflect key parameters of bond strength such as cover, bar spacing and presence of transverse reinforcement. These parameters become more critical in the case of splitting failures. The presence of epoxy-coating on the bars diminishes the adhesion and friction components of the mechanism of bond strength; thus, making bearing of the deformations the main component in the bond strength of epoxy-coated deformed bars. The ACI 318-05 development length equation will be used in the design of splice lengths for the test specimens in this study because it does contain such parameters. However, both sets of specifications will be evaluated with respect to the experimental results.

The experimental data reported in Chapter 4, is analyzed in Chapter 5 to determine if the current specifications in AASHTO LRFD (2004) and ACI 318-05 need to be modified to allow up to 18 mils of epoxy-coating thicknesses. In Chapter 6, the impact to the design specifications is discussed and a field instrumentation plan for a future concrete bridge deck containing extra-coating thickness bars is outlined.

Figure 2.1: Typical bond stress vs. slip of steel reinforcing bars (Borosnyoi and Balazs 2003)

Figure 2.2: Cracking and damage mechanisms in bond: (a) side view of a deformed bar with deformation face angle α showing formation of Goto (1971) cracks; (b) end view showing formation of splitting cracks parallel to the bar; (c) end view of a member showing splitting crack between bars and through the concrete cover; and (d) side view of member showing shear crack and/or local concrete crushing due to bar pullout (ACI 408R-03)

Figure 2.3: Components of bond with and without friction (Treece and Jirsa 1989)

Figure 2.4: Bond comparison of epoxy-coated bars, which are clean of concrete after failure, to uncoated bars (web reinforcement), which have concrete adhered to the surface

Figure 2.5: Epoxy-coated reinforcement and surrounding concrete after splice failure

Figure 2.6: Schematic view of flat plate friction test (Cairns and Abdullah 1994)

Figure 2.7: Typical shear stress versus relative slip relationship for a flat-plate friction specimen (normal stress $f_n = 9 \text{ N/mm}^2$) (Cairns and Abdullah 1994)

$$R_r = \frac{\text{Bearing area}}{\text{Shearing area}} \approx \frac{h_r}{s_r}$$

(Actual R_r values range from 0.8 to $0.9 \frac{h_r}{s_r}$)

Figure 2.8: Definition of relative rib area, R_r (Darwin, Lutz, and Zuo 2005)

Figure 2.9: Schematic of: (a) pullout specimen; (b) beam-end specimen; (c) beam anchorage specimen; and (d) splice specimen (ACI 408R-03)

Figure 2.10: Load-slip curves for No. 5 bars (Choi, Hadje-Ghaffari, Darwin, McCabe 1991)

Figure 2.11: Reduced rib height due to epoxy-coating (Grundhoffer, Mendis, French, and Leon 1998)

No. 8 bars

No. 6 bars

No. 5 bars

Figure 2.12: Relative bond strength C/U versus coating thickness for No. 8 bars, No. 6 bars, and No. 5 bars (Choi, Hadje-Ghaffari, Darwin, McCabe 1991)

3 EXPERIMENTAL PROGRAM

3.1 Introduction

This chapter covers the experimental program which includes the objective and description of test program, design of test program, material properties, instrumentation, construction, test setup, test protocols and data collection.

3.2 Objective of Experimental Program

The objectives of this research were to evaluate the applicability of the equations from the current specifications for bond development length (ACI 318-05 (12-1) and AASHTO LRFD 2004 (5.11.2.1.1)) up to 18 mils of epoxy-coating thickness (on No. 5 bars and No. 9 bars) and to study the effect of epoxy-coating thickness on the bond strength for No. 5 and No. 9 bars.

3.2.1 Test Program

Twenty splice specimens were tested using the setup illustrated in Figure 2.9d. The test program for thicker epoxy-coating reinforcement is shown in Table 3.1. The specimens are grouped into four groups in accordance to research objectives: Specimens 1A to 4B study the effect of epoxy-coating thickness on bundled bars (two 3-bar bundles per specimen; one splice per bundle; two continuous bars per bundle) for No. 5 and No. 9 bars; Specimens 5A to 6B were aimed at evaluating the current specification (splice length designed using the ACI 318-05 (12-1) equation for development length; three splices per specimen) for only 18-21 mils of epoxy coating thickness for No. 5 and No. 9 bars; Specimens 7A to 8B study the effect of epoxy-coating thickness for No. 5 bars (three single splices per specimen); and Specimens 9A to 10B were directed to study the effect of epoxy-coating thickness for No. 9 bars (three single splices per specimen). Two different ranges of epoxy-coating thicknesses were evaluated, 12-15 mils and 18-21 mils. Other properties common to all specimens were: Class A concrete, with minimum specified compressive strength of 4000 psi, 4 in. slump, and No. 8 crushed lime stone; 2 in. concrete cover; 12 in. height, 20 in. width, and 14 feet length of specimen. Different parameters that affect the bond strength are held constant so that the main parameter of thickness of epoxy-coating may be studied. The top bar effect due to the casting position (concrete below spliced reinforcement during casting is greater than 12 inches) was not studied. All specimens did not have transverse reinforcement within the splice region. The last two constraints were the result of focusing the study to concrete bridge decks.

3.2.2 Design of Test Program

Several studies were considered in the design decisions made with respect to the test program. The Miller, Kepler and Darwin (2003) study (using beam-end specimens (Figure 2.9b)) concluded that a maximum of 16.5 mils of epoxy-coating thickness could

be used for No. 6 bars and greater; however, the Samples and Ramirez (2000a,b) studies indicated the need to increase the coating thickness beyond the limit stated by Miller et al. (2003). The range of 18-21 mils was therefore specified for the upper range studied due to the Samples and Ramirez (2000a, b) studies. The bond performance would then be compared with bars having epoxy-coatings in the range of 12-15 mils, which is below the Miller et al. (2003) limit of 16.5 mils. Also in the Miller et al. (2003) study, No. 6 bars and greater were valid for the epoxy-coating thickness limit; however epoxy-coated No. 5 bars are also used during construction of concrete bridge decks. Therefore No. 5 bars were chosen as the smaller bar size used in this study, and No. 9 bars were chosen as the larger bars used for comparison. Splice specimens (as opposed to beam-end specimens used in the Miller et al. (2003) study) were used because the current specifications for bond development length are based on results from splice specimen tests and the goal of the study was to determine if the specifications could be extended up to 18 mils of thickness of epoxy-coating.

The Jirsa, Chen, Grant and Elizondo (1995) study looked at the effect of splices in bundled bars, with mainly uncoated bars studied. Only two bundled bar tests (2-bar bundles and two layers of reinforcement) were carried out with epoxy-coated bars, which also included epoxy-coated transverse reinforcement in the splice region. Epoxy patching material was applied to the No. 6 bundled bars and the No. 4 stirrups used in the test region. The coating thicknesses for these bars were in the range of 3 to 9 mils, which were measured using a Microtest thickness gage. These coated bundles reached stresses equal to or greater than the uncoated bundled bars tested; therefore, it was stated that there was no concern regarding coated bundled bars when confined by adequate transverse reinforcement. Due to the limited knowledge of the effect of epoxy-coating for bundled bars without transverse reinforcement in their splice length, this study also looked at the effect of epoxy-coating thickness for 3-bar bundles without transverse reinforcement present in the splice region.

The splice lengths were designed using ACI 318-05 Equation (12-1) (Equation 2.5 in this report) with two stress levels, f_y and $0.5f_y$. The specimens designed with a steel stress equal to f_y are intended to evaluate ACI 318-05 Equation (12-1) for development length. Half the yield strength is also used because the bond strength is based on the *force* the bond length is able to develop, and not the *strain*. Therefore, these tests were designed to keep the bar stress in the linear elastic range to allow for comparison of bars with different bar diameters and different epoxy-coating thicknesses. Also, 10 in. was set as a minimum splice length which affected the design stress for Specimens 7A to 8B. The design stress used for Specimens 7A to 8B was also used for Specimens 9A to 10B so that the effect of the bond strength due to rebar size may also be compared. The effect of epoxy-coating thickness on bundled bars (using No. 5 bars and No. 9 bars) is also evaluated by designing the splice using ACI 318-05 (12-1) with stress level of $0.5f_y$.

Table 3.2 summarizes the specimen design. The selected length of lap-splice (ℓ_a) is used to determine the calculated maximum stress (f_s) in the reinforcement for each specimen, which is calculated by using the following modified form of ACI 318-05 Equation (12-1) (Equation 2.5 in this report):

$$f_s = \left(\frac{40}{3} \sqrt{f'_c} \frac{\left(\frac{c_b + K_{tr}}{d_b} \right)}{\psi_t \psi_e \psi_s \lambda(B.B.F.)} \right) \frac{\ell_a}{d_b} \quad (3.1)$$

where the majority of the factors are defined in Section 2.3.2, and the values used for the development factors are listed in Table 3.2. *B.B.F.* is the bundled bar factor, which accounts for the specified increase in development length for bundling of bars as stated in ACI 318-05 12.4. The compressive strength used in the design (f'_c) was 6000 psi, which is the average compressive strength of a sample batch of Class A concrete (minimum specified compressive strength of 4000 psi). The Cast Number refers to the casts discussed in the concrete material properties section (Section 3.3.1) and in Table 3.3, Table 3.4, and Figure 3.1.

3.3 Material Properties

This research project studies the interaction of three main materials: concrete, steel reinforcing bars, and epoxy-coating on the steel reinforcing bars. This section discusses the material properties for the materials used in each specimen.

3.3.1 Concrete

Concrete use in this research project was batched and delivered by Irving Materials Inc. (IMI) from their West Lafayette, Indiana location. The mix composition for each concrete cast is summarized in Table 3.3. Each cast is a five cubic yard batch. Unless otherwise noted, the amounts for the different components listed in this table for each cast were provide by Irving Materials Inc. except for the following: the Actual Water (lb) was calculated from the volume of water (Actual Water (gal)) and the Slump (in.) was measured using a slump cone, in accordance to ASTM standard practices, at the time of arrival to the cast location (Bowen Laboratory, Purdue University).

The compressive strength of the concrete for each cast was found by testing 4 in. by 8 in. concrete cylinders in accordance to ASTM C 39. An average of three cylinder breaks is one compressive strength for a given age. The flexural strength of the concrete for each cast was found by testing 6 in. square cross-section concrete rupture beams in third-point loading in accordance to ASTM C 78. The average concrete compressive and flexural strength for each cast are summarized in Table 3.4. Compressive strength tests were taken at different ages, and each splice specimen is associated with its test date compressive strength. The average of the test date compressive strengths for a given cast (four splice specimens per cast) is the Average Test Compressive Strength. The average of at least two rupture beams tested around the time that the splice specimens were tested is the Average Flexural Strength. A record of all the compressive strength tests taken at different ages is presented in Figure 3.1. These are strength gain curves where Compressive Strength is plotted versus its corresponding age, or Time After Cast. The

date that the first splice specimen tested in each cast is represented by the dotted line labeled First Test Date.

3.3.2 *Steel Reinforcing Bars*

Two different Grade 60 rebar sizes are used, No. 5 bars and No. 9 bars. All bars of the same size are the same heat, and therefore each bar size has the same reinforcement properties as summarized in Table 3.5. The Stress vs. Strain curves from tension tests are presented in Figure 3.2 for both bar sizes. The data from Bar 5-1 and Bar 9-1 in Figure 3.2 (approximate averages of the tensile tests) are used with the 0.2% offset (2000 $\mu\epsilon$) of the Stress vs. Strain curve to find the Yield Strength for each bar size respectively. Figure 2.8 shows the schematic of the following reinforcement properties, which were calculated with epoxy-coating present on the steel reinforcing bars: the Average Rib Height (h_r); the Rib Spacing (s_r); the Rib Gap (Gap), which is the thickness of the longitudinal rib; and the Deformation Angle, which is the angle of the ribs from the longitudinal axis. The Relative Rib Area (R_r) was calculated using the ACI 408R-03 (6-1) equation:

$$R_r = \frac{h_r}{s_r} \left(1 - \frac{\sum gaps}{p} \right) \quad (3.2)$$

where two rib gaps along the longitudinal length of each bar need to be accounted for in $\sum gaps$ and p is the nominal perimeter of bar, which is $\pi(Nominal\ Bar\ Diameter)$. The Elongation is found from using ASTM A 370 tension test procedure by using an 8 in. gage length and fitting the broken ends together to measure the increase in length of the gage length.

3.3.3 *Epoxy-Coating*

Epoxy-coating thickness measurements were recorded at two different times. First, surveys of epoxy-coating thicknesses were recorded for all the bars supplied. Then when the bars were selected and cut to the appropriate length for the specimens, epoxy-coating thickness measurements were recorded for every portion of epoxy-coated bar in the splice-region.

3.3.3.1 *Survey of All Bars Supplied*

Epoxy-coating thickness measurements (Table 3.6) were taken on March 8, 2004 for all the epoxy-coated bars coated and supplied by R. J. Rebar (now Gerdau Ameristeel's Muncie Rebar Coating Plant). There were four groups consisting of fifteen No. 9 bars with 18-21 mils, twelve No. 9 bars with 12-15 mils, seven No. 5 bars with 18-21 mils, and five No. 5 bars with 12-15 mils delivered. The Bar Number's notation seen in the table (Table 3.6) consists of "Size of Bar"- "Bar Number"/ "Lowest Coating Thickness for the Range Specified." An ending is added to the notation (TOP or BOT) if only one side's statistics are recorded for reasons of comparing side to side statistics. For instance,

9-1/18 BOT stands for No. 9 bar, bar 1 measurements, 18 mils is the lowest coating-thickness specified for that batch and BOT indicates that measurements are only from bottom bar portion.

ASTM A 775M Section 8.1 was followed and the readings were taken with an Elektro-Physik Minitest 3001 coating thickness gauge that digitally displays three significant digits. Epoxy-coating thickness readings were taken at fifteen equally spaced locations along the 60 ft long bar with a reading taken at the top and bottom at each location, therefore a total of thirty readings per bar were taken in all bars. The readings were taken in between the deformations as indicated in A 775M. The average, standard deviation, maximum, and minimum for each bar's coating thickness readings were recorded, as shown in the table. The cumulative values for each bar are shown in the shaded rows of the appended tables.

Some trends of thicknesses as indicated by the measurements taken on these bars are noted. The average thickness per bar, for the majority of the bars, is within the specified range. Some bars seemed to have different coating thicknesses on its opposing side, therefore fifteen readings were taken on the top (TOP) and its statistics were recorded to compare to fifteen more reading which were taken from the bottom (BOT) of the bar. It can be seen that the bars with different coating thicknesses on the top versus the bottom have higher standard deviations than the bars that did not share this trend. If we look at standard deviations for each group, it can be seen that the lowest average standard deviations are for the 12-15 mils No. 5 bars, and the highest average standard deviations are for the 18-21 mils No. 9 bars.

3.3.3.2 *Splice-Region Measurements*

Epoxy-coating thickness readings, shown in Table 3.7, were recorded for every portion of epoxy-coated bar in the splice-region. Readings were taken on both sides of the bar approximately every 5 in. within the splice region, and a mean, standard deviation (S.D.), maximum (Max), and minimum (Min) epoxy-coating thickness readings were recorded for both the top of the ribs and the valleys (between the ribs). The specimen notation is associated with the specimen number (No.) in Table 3.1, the strain gage notation is associated with the location of the bar in the specimen by the strain gage identification as described in Section 3.4 and shown in Figure 3.3, the coating thickness is the minimum epoxy-coating thickness for the range specified (12 mils for the 12-15 mils range, and 18 mils for the 18-21 mils range), and the "From Bar" notation is the origin of the bar from the original 60 ft bar surveyed as discussed in Section 3.3.3.1 and associated with the numbering in Table 3.6. For instance in specimen 6A, bar location with respect to strain gage 3S, minimum coating thickness of 18 mils specified, and originating from bar 9-3/18, following mean readings for thicknesses of epoxy-coating are as follows: 23.5 mils on top of the rib and 20.6 mils in the valley between the ribs, where a range of 18-21 mils was requested. It was observed from Table 3.7 that the mean coating thickness readings on the Ribs were always greater than in the Valleys.

3.4 Instrumentation

The splice specimens tested in this research project were instrumented to record three different measurements during a test: the strain in the epoxy-coated reinforcement and the compression zone in the concrete, the vertical deflection along the length of the beam, and the load applied to the ends of the beam. The typical location of the instrumentation is summarized in Figure 3.3 for both the Bundled Bar and the Single Splice specimens, unless otherwise noted.

3.4.1 Epoxy-Coated Reinforcement Strain Gages

The strain in the epoxy-coated reinforcement is measured using type CEA-06-250UN-120 strain gages from Vishay Micro-Measurements Group, Inc. These steel strain gages are located 3 in. away from the splice-region, unless otherwise noted. The typical exceptions are for strain gages 3S and 3N in the typical Bundled Bar specimen where they are located 1.5 in. away from 2S and 1N respectively (or 4.5 in. away from the splice-region) and strain gage 4N (located on the same bar that strain gage 2N for the Bundled Bar or strain gage 3N for the Single Splice) where it is located in the middle of the Support and Load points. The exceptions to the typical Strain Gage Location are both Specimen 1A and Specimen 4A have the 2N (and 4N) and 1N typical strain gage locations switched. The typical strain gage locations should also be noted for the Bundled Bar specimens, using the cross-section view of the splice-region (Figure 3.4). Typically in the splice-region, the top spliced bars in the bundle are strain gage 1S and 2N, the side (exterior) spliced bars in the bundle are strain gage 2S and 1N, the side (interior) continuous bars in the bundle are strain gage 3S and 3N, and the bottom continuous bars are not instrumented with a strain gage. The exceptions to the typical cross-section strain gage location are as follows: Specimen 1A and Specimen 4A where the top spliced bars in the bundle are strain gage 1S and 1N and the side (exterior) spliced bars in the bundle are strain gage 2S and 2N and also Specimen 1A the side (interior) continuous bars in the bundle are not instrumented with a strain gage, and the bottom continuous bars are strain gage 3S and 3N. The wires attached to the strain gages were guided along the steel reinforcement to the closest support location where all wired merged together at an output port at each support location. The wires were then connected to the channels of the Data Acquisition System discussed in Section 3.4.5.

3.4.2 Concrete Compression Zone Strain Gage

The strain in the concrete compression zone is measured using type EA-06-40CBY-120 strain gages from Vishay Micro-Measurements Group, Inc., which is bonded to the exterior of the concrete beam. This concrete strain gage 4S is located on the west side of the beam in the middle of the splice-region, unless otherwise noted. Exceptions to the typical Strain Gage Location are as follows: Specimens 1A, 1B, 2A, and 2B have an extra concrete strain gage 5S located on the west side of the beam (where strain gage 4S is typically located) and strain gage 4S is moved to the east side of the beam (still in the middle of the splice-region) and also Specimen 1B has an extra concrete strain gage 5N

located to the north of strain gage 5S on the west side. The concrete gage is placed as close to 0.75 in. away from the bottom of the concrete beam.

3.4.3 Linear Variable Differential Transformers (LVDTs)

The vertical deflection of the concrete beam was measured at discrete points along the beam using LVDTs. The nine LVDTs were all Lucas Schaevitz DC-operated transformers with specific information for each LVDT listed in Table 3.8. The location of the LVDTs along the length of the beam, which are shown in Figure 3.3, have their specific location listed in the table, Distance from Center, by using the centerline of the beam as a datum point. A positive distance from center is to the north, and a negative distance from center is to the south. LVDT 5S and LVDT 5N measure the support displacements, and LVDT 4N measures the center of the beam displacement. The Model Number and Range, or displacement capacity, are specified by the manufacturer. Each LVDT was calibrated using a Boeckeler micrometer. The LVDTs were mounted to a steel frame that was supported by the laboratory floor and was independent of the test specimen, load frames, and supports. The LVDT cores were attached to the beam by using 3/16 in. threaded rods and nuts that created a connection to small steel angles attached to the concrete beam with epoxy glue.

3.4.4 Load Cells

The load applied to the ends of the beam was measured using load cells. Three (of the four total) load cells were Lebow load cells, and the fourth load cell was a Tokyo Sokki Kenkyujo Co., Ltd. (Tokyo Sokki) load cell. Table 3.9 lists specific information for each load cell, where the Model, Capacity, and Serial Number were provided by the manufacturers. The location of the load cells, described in Table 3.9, lie along the Load line shown in Figure 3.3.

3.4.5 Data Acquisition

The instrumentation measurements were acquired during tests with Vishay Micro-Measurements Group's System 5000 and a personal computer running StrainSmart, software provided by Vishay Micro-Measurements Group. Two System 5000's scanners (Model 5100) were used. The first scanner housed four strain gage cards (five channels per card) with the North strain gage channels on the first card, South strain gage channels on the second card, Load Cell channels on the third card and no channels used on the fourth card. The second scanner housed four high-level input cards (five channels per card) with the North LVDT channels on the first card, South LVDT channels on the second card and no channels used on the third and fourth card.

3.5 Construction

The construction of the splice specimens started with the epoxy-coated bars. The epoxy-coated bars were coated by R. J. Rebar (now Gerdau Ameristeel's Muncie Rebar Coating Plant) with fusion bonding flexible coating, which was a 3M and Valspar powder coating mixture. After these bars were cut to appropriate lengths, instrumented with strain gages, and measurements taken of epoxy-coating thicknesses, they were assembled into rebar cages as shown in Figure 3.5. The cages consisted of the epoxy-coated longitudinal bars on the top row (bar size and configuration varies depending on the specimen) and two No. 6 black bars on the bottom row, which is shown in the splice-region cross-section view, Figure 3.4. All black steel was supplied by J & K Supply Inc. (Lafayette, Indiana). The longitudinal bars were attached (using wire ties) to the black steel stirrups (No. 3 bars), which were only present in the beam from the support to the end of the beam. These stirrups were used as transverse reinforcement to help provide shear resistance in this region so that the specimen will not fail in shear before the splice region fails.

Formwork for the 14 ft. long 12 in. by 20 in. concrete beams was built using 3/4 in. thick plywood sheets and construction grade 2 in. by 4 in. wood. Form oil was used as a release agent and was sprayed on the formwork before the 2 in. steel chairs were placed on the bottom of the formwork. The reinforcement cage was then placed in the formwork on top of the chairs, as shown in Figure 3.6. This placement was done so that the top and side cover of the epoxy-coated reinforcement in the splice-region was as close to 2 in. as possible, as shown in Figure 3.4. The longitudinal black steel reinforcement location depends on whether the stirrups or the longitudinal black steel was placed on top of the 2 in. steel chairs located at the bottom of the beam. Specimens 1A through 2B had the stirrups placed on top of the 2 in. steel chairs; therefore the effective depth for the longitudinal black steel reinforcement for these specimens is 2.75 in. from the bottom of the beam. Specimens 3A through 10B had the longitudinal black steel placed on top of the 2 in. steel chairs; therefore the effective depth for the longitudinal black steel reinforcement for these specimens is 2.375 in. from the bottom of the beam. To prevent the reinforcement cage from moving around during the cast, the reinforcement cage was secured to the formwork using wire ties. At this point, all the as-built measurements (discussed later in this section) were recorded. The cast setup, shown in Figure 3.7, was then organized by setting up the thirty 4 in. by 8 in. cylinder molds, slump test, and the six flexural beam molds with the four splice specimens ready to be cast. The flexural beam molds also had release agent sprayed in them. IMI delivered the Class A concrete, the slump test was taken according to ASTM C 143, and the concrete was cast directly into the forms for the splice specimens in two lifts, as shown in Figure 3.8. The flexure beams and cylinders were filled by scooping concrete from wheelbarrows into these molds. Vibrators were used to consolidate the concrete in the splice specimens and flexure beams, and tamping rods were used to consolidate the concrete for the cylinders and the slump test (as required by ASTM C 143).

After casting, the concrete specimens (including the cylinders and flexure beams) were covered with wet burlap and plastic for wet curing. The burlap was kept wet during this curing time. The specimens were typically cured for seven days, except for Cast 3 was cured for twenty-nine days due to lower compressive strength tests at early ages. All the

forms were stripped off the specimens after wet curing was done, and then they were air-cured until their respective test date. No splice specimen was tested until after its age of 28 days.

The as-built data for all the splice specimens is listed in Table 3.10. The total height of the beam is h . The effective depth of the beam is d , which the calculation for this is discussed later in this paragraph. The width of the bottom of the beam at the section where the concrete strain gage is located is b_{bottom} . The Splice Location refers to the typical layout shown in Figure 3.3, where there are West (top) and East (bottom) splices for the Bundled Bar specimens and there are West (top), Center (middle) and East (bottom) splices for the Single Splice specimens. For the Single Splice specimens, the Center-W and Center-E notations differentiate the clear spacing on the West (top) side and the East (bottom) side of the Center splice, respectively, for the Side Cover measurements. The measured length of each splice is ℓ_a . The Side Cover and Top Cover notations (S, S-C, C, N-C and N) refer to the Cover Measurement Section shown in Figure 3.3, where S and N are located at the South and North support, or the first stirrup in the rebar cage, S-C and N-C refer to the South and North ends of the splice, and C refers to the center of the splice. The highlighted Splice Location refers to the Failed Splice Location noted in Table 4.2. The data for the failed splice is used to calculate d for each specimen in the following way:

$$d = h - (\text{Effective Top Cover}) - 0.5d_{\text{bEffective}} \quad 3.3)$$

where the Effective Top Cover and $d_{\text{bEffective}}$ are defined below:

- Bundled Bar specimens (1A to 4B) use the average of all the shaded measurements in Top Cover's S and N columns as the Effective Top Cover; a unit of bundled bars (three bars per bundle) is treated as a single bar of a diameter ($d_{\text{bEffective}}$) derived from the equivalent total area (described in ACI 318-05 12.4.2).
- Specimens 5A to Specimen 6B use the average of all the shaded measurements in Top Cover's C column as the Effective Top Cover; the nominal bar diameter of a single bar is $d_{\text{bEffective}}$.
- Specimens 7A to Specimen 10B use the average of all the shaded measurements in Top Cover's S-C and N-C columns as the Effective Top Cover; the nominal bar diameter of a single bar is $d_{\text{bEffective}}$.

The concrete clear cover used in Chapter 5 (Section 5.3) to determine the calculated values of stress according to Code Specifications is the smallest highlighted concrete cover value, Side Cover or Top Cover, in Table 3.10. For Bundled Bar specimens, this highlighted concrete cover value was restricted to the center of the splice location (C) for Side and Top Cover to use the smallest *average* cover of the splice-region due to the bundling configuration causing the spliced bars to slope out more towards the end of the splice.

3.6 Test Setup

The test setup for all the specimens is shown in Figure 3.9. The concrete specimen was placed on two supports (pin connection at right and roller connection at left) and load is applied to the specimen at both ends with pin connections. Between the supports, a constant moment region is created, which is where the lap-splice is located (ℓ_a = length of lap-splice). Transverse reinforcement is used in the constant shear region between the load point and the support to prevent shear failures prior to splice failure (Figure 3.5).

There are three variations of the loading point test setup, as shown in Figure 3.10 through Figure 3.12. Various loading system configurations were used in an attempt to eliminate excessive bending in the anchored threaded rods at high loads. Each variation was an attempt at decreasing this effect. In general, hydraulic rams are used to apply load at the ends of the blue steel beam by reacting against anchored threaded rods. The steel beam transfers the applied force to the pin connection, which transfers the force to the concrete specimen. A hydraulic hand pump is used to control the load applied by the hydraulic rams by applying the same pressure to the rams. The labels in these figures point out the objects that contribute to the self-weight of the loading point test setup or that are not fixed in relation to the anchored threaded rods. The calculations for the loading point test setup self-weight, the Test Setup Load, is shown in Table 3.11. The specimens are ordered according to their Test Date. The Equipment for Test Setup Load lists the objects that contribute to the Test Setup Load, which is determined by the objects the Load Cell does not account for at the time the load is zeroed. The sum of the weight of these objects is the Test Setup Load. The figure referred to in parentheses is the figure for the appropriate Loading Point Test Setup at Failure.

3.7 Test Protocols and Data Collection

Before each test, all sensors were zeroed and an initial reading was recorded using a personal computer that controlled the data acquisition system discussed in Section 3.4.5. During each test, small increments of load were manually applied using the hydraulic hand pump. Sensor readings were recorded and new crack patterns were marked at each load increment after the applied load stabilized for at least 5 seconds. Load increments depended on the bar size used in the specimen because specimens with larger bars fail at higher loads. Specimens with No. 5 bars had loading increments of 1000 lb., and specimens with No. 9 bars had loading increments of 2000 lb. Pictures of the crack pattern were taken and crack widths were recorded every few loading increments. When the beam approached failure, sensor readings were recorded more frequently to capture the failure response of the beam.

The following data was monitored:

- Vertical load applied at the ends of the concrete specimen using load cells
- Vertical deflection using Linear Variation Differential Transformers (LVDTs)
- Strain in tension reinforcement using strain gages

- Strain of concrete in flexural compression region using external surface concrete strain gages

The following was manually recorded:

- Crack pattern at various load levels marked with permanent marker
- Crack widths of two specified cracks per specimen using a crack comparator

3.8 Summary

The different aspects of the experimental program, design, material properties, instrumentation, construction, test setup, test protocols and data collection, were described in this chapter. The test data from the program are presented in Chapter 4, and analyzed in Chapter 5.

No.	Description (bar type, coating thickness, designed f_s , beam height)	Test Dates	Research Objective
1A	No. 5 bundle*, 12 mils, $1/2 f_y$, 12 in.	08/31/2004	Effect of Coating Thickness on Bundled Bars
1B		6/16/04 - 6/18/04	
2A	No. 5 bundle*, 18 mils, $1/2 f_y$, 12 in.	6/24/04 - 8/4/04	
2B		09/09/2004	
3A	No. 9 bundle*, 12 mils, $1/2 f_y$, 12 in.	10/7/04 - 12/3/04	
3B		01/24/2005	
4A	No. 9 bundle*, 18 mils, $1/2 f_y$, 12 in.	12/10/2004	
4B		12/21/2004	
5A	No. 5, 18 mils, f_y , 12 in.	02/09/2005	Evaluate Current Specification for 18 mils
5B		02/17/2005	
6A	No. 9, 18 mils, f_y , 12 in.	03/03/2005	
6B		03/08/2005	
7A	No. 5, 12 mils, $1/2 f_y$, 12 in.	04/30/2006	Effect of Coating Thickness on No. 5 Bars
7B		05/30/2006	
8A	No. 5, 18 mils, $1/2 f_y$, 12 in.	05/12/2006	
8B		05/19/2006	
9A	No. 9, 12 mils, $1/2 f_y$, 12 in.	09/28/2005	Effect of Coating Thickness on No. 9 Bars
9B		10/28/2005	
10A	No. 9, 18 mils, $1/2 f_y$, 12 in.	11/10/2005	
10B		12/21/2005	

*Bundle is a three-bar bundle

Table 3.1: Test Program for Thicker Epoxy-Coating Reinforcement

Specimen No.	Bar Size	Designed Failure Stress	ℓ_a (in.)	*Cast Number	Development Factors				
					ψ_t	ψ_e	ψ_s	λ	B.B.F.
1A, 1B, 2A, 2B	No. 5	$f_s = 38$ ksi, 3 bar bundle	14	1	1.0	1.5	0.8	1.0	1.2
3A, 3B, 4A, 4B	No. 9	$f_s = 33$ ksi, 3 bar bundle	43	2	1.0	1.5	1.0	1.0	1.2
5A, 5B	No. 5	Code f_y ($f_s = 60$ ksi)	14	3	1.0	1.2	0.8	1.0	1.0
6A, 6B	No. 9	Code f_y ($f_s = 60$ ksi)	43	3	1.0	1.5	1.0	1.0	1.0
7A, 7B, 8A, 8B	No. 5	$f_s = 43$ ksi	10	5	1.0	1.2	0.8	1.0	1.0
9A, 9B, 10A, 10B	No. 9	$f_s = 43$ ksi	31	4	1.0	1.5	1.0	1.0	1.0

*Cast Number is related to the Cast in Table 3.3, Table 3.4, and Figure 3.1.

Table 3.2: Specimen Design

Components	Cast				
	1	2	3	4	5
No. 8 Crushed Limestone (lb)	8740	8720	8740	8920	8840
Sand (lb)	7000	7140	7080	6940	6900
Cement (lb)	2810	2810	2800	2810	2960
Water (lb)	968	1052	1012	938	959
Sand Moisture (%)	4.00	4.00	4.00*	4.00*	4.00*
Water from Sand (gal)	32.26	32.91	33.94*	33.27*	33.07*
Actual Water (gal)	148.3	159	155*	146*	148*
Actual Water (lb)	1238	1327	1295*	1216*	1235*
Air (oz)	16.5	15.5	10	20	11
Reducer (oz)	42	42	113	56	119
w/c	0.445	0.472	0.463*	0.433*	0.417*
Slump (in.)	2.75	4.75	5.5	3.5	4.25
Date	05/14/2004	06/07/2004	09/15/2004	06/15/2005	08/22/2005

*Values estimated assuming Sand Moisture = 4.00%

Table 3.3: Mix Composition for Concrete Casts

Cast	*Average Test Compressive Strength (ksi)	**Average Flexural Strength (psi)
1	5.5	788
2	4.9	726
3	5.8	753
4	5.4	674
5	5.1	866

*4"x8" cylinders

**Data reported for average value for tests; rupture beams tested at approximately the same time the splice beam specimens were tested

Table 3.4: Average Concrete Compressive and Flexural Strength Summary

Deformation Pattern	Nominal Bar Diameter (in.)	Yield Strength (ksi)	Average Rib Height* (in.)	Rib Spacing (in.)	Rib Gap (in.)	Deformation Angle (degrees)	Relative Rib Area	Elongation (%)
No. 5	0.625	62.2	0.031	0.411	0.14	70	0.064	8.2
No. 9	1.128	76.7	0.064	0.687	0.15	60	0.085	15.8

*Average height of deformations h_r is determined from measurements made on not less than two typical deformations on each side of bar. Determinations are based on five measurements per deformation: one at center of overall length, two at ends of overall length, and two located halfway between center and ends. The measurements at ends of overall length are averaged to obtain a single value and that value is combined with the other three measurements to obtain average rib height h_r (Miller, Kepler and Darwin, 2003).

Table 3.5: Reinforcement Properties

18-21 mils, No. 9 Bars

Bar No.	Readings	Average	Stand. Dev.	Max.	Min.
9-1/18	28	20.9	2.51	25.1	16.8
9-1/18 TOP	15	19.0	1.29	20.7	16.0
9-1/18 BOT	15	23.7	2.28	27.8	18.8
9-2/18	30	19.6	2.71	24.2	14.7
9-3/18	30	21.3	3.12	26.7	15.8
9-3/18 TOP	15	18.8	1.81	21.6	15.0
9-3/18 BOT	15	23.7	2.29	26.2	18.9
9-4/18	29	22.2	1.60	25.3	18.3
9-5/18	30	19.0	2.09	23.8	14.3
9-6/18	30	23.8	2.70	28.7	18.4
9-7/18	30	21.8	2.62	25.7	17.1
9-7/18 TOP	15	19.5	1.44	23.4	17.6
9-7/18 BOT	15	21.9	1.58	25.0	19.0
9-8/18	30	20.0	2.11	25.5	16.4
9-9/18	30	23.7	2.06	27.1	18.7
9-10/18	28	19.2	2.56	24.8	14.0
9-10/18 TOP	14	21.0	2.34	25.0	16.7
9-10/18 BOT	14	16.8	1.86	20.1	13.9
9-11/18	30	20.6	1.93	24.5	17.1
9-12/18	30	19.2	2.68	24.1	14.3
9-13/18	30	24.3	3.28	30.3	19.0
9-13/18 TOP	15	26.4	2.26	29.7	22.4
9-13/18 BOT	15	21.0	1.37	23.3	18.6
9-14/18	30	20.8	1.96	24.3	17.2
9-15/18	30	18.7	1.46	22.8	15.7

12-15 mils, No. 9 Bars

Bar No.	Readings	Average	Stand. Dev.	Max.	Min.
9-1/12	30	17.7	1.68	22.7	14.9
9-2/12	30	16.6	1.35	20.0	14.6
9-3/12	30	13.9	1.61	16.9	10.3
9-3/12 TOP	15	12.8	1.16	14.5	10.1
9-3/12 BOT	15	14.4	1.38	17.0	12.5
9-4/12	30	17.0	1.16	20.4	14.1
9-5/12	30	13.0	1.71	17.9	9.1
9-6/12	30	16.1	1.46	18.8	12.6
9-7/12	30	14.4	2.38	17.9	10.4
9-7/12 TOP	15	12.3	1.61	16.2	10.0
9-7/12 BOT	15	16.8	0.81	18.1	15.3

Table 3.6: Epoxy-Coating Thickness Measurements (between-ribs along two sides of each bar supplied) for No. 9 Bars (18-21 mils and 12-15 mils) and No. 5 Bars (18-21 mils and 12-15 mils)

12-15 mils, No. 9 Bars (cont.)

Bar No.	Readings	Average	Stand. Dev.	Max.	Min.
9-8/12	30	13.8	2.72	19.1	9.4
9-8/12 TOP	15	15.9	1.70	18.9	13.3
9-8/12 BOT	15	11.4	1.37	14.2	9.3
9-9/12	30	16.5	0.79	18.0	14.7
9-10/12	30	13.2	1.53	17.4	10.0
9-11/12	30	14.1	2.13	18.8	11.2
9-12/12	30	11.9	2.29	16.3	8.5
9-12/12 TOP	15	10.4	1.40	13.7	8.6
9-12/12 BOT	15	13.7	2.07	17.0	10.6

18-21 mils, No. 5 Bars

Bar No.	Readings	Average	Stand. Dev.	Max.	Min.
5-1/18	30	21.0	2.24	25.0	17.2
5-2/18	30	21.9	1.65	24.2	16.7
5-3/18	30	20.0	2.11	25.6	16.0
5-4/18	30	22.0	1.40	23.8	17.9
5-5/18	30	20.6	1.28	23.9	18.7
5-6/18	30	20.7	2.03	25.1	17.2
5-7/18	30	20.3	1.77	23.4	17.2

12-15 mils, No. 5 Bars

Bar No.	Readings	Average	Stand. Dev.	Max.	Min.
5-1/12	30	12.8	1.16	15.4	9.7
5-2/12	30	13.8	0.88	15.4	11.9
5-3/12	30	13.6	1.28	16.5	11.5
5-4/12	30	13.9	0.90	15.3	11.7
5-5/12	30	13.6	1.08	15.5	11.1

Table 3.6 (cont.): Epoxy-Coating Thickness Measurements (between-ribs along two sides of each bar supplied) for No. 9 Bars (18-21 mils and 12-15 mils) and No. 5 Bars (18-21 mils and 12-15 mils)

No. 5 Bundled Bars - $\ell_a = 14"$ (3 readings each side, therefore 6 readings for Valley and Rib)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
1A	12 mils	1N	5-5/12	15.3	1.62	17.4	13.7	17.1	0.77	18.3	16.0
		1S	5-5/12	14.9	1.13	16.0	13.5	17.3	0.53	18.0	16.4
		2N & 4N	5-5/12	14.6	1.17	16.7	13.5	17.3	0.59	18.2	16.5
		2S	5-4/12	13.9	0.65	14.6	12.8	16.1	0.37	16.5	15.6
1B	12 mils	1N	5-4/12	13.2	0.84	14.0	11.8	16.2	0.80	16.8	14.7
		1S	5-4/12	13.3	0.72	14.4	12.3	16.4	0.70	17.3	15.2
		2N & 4N	5-4/12	12.3	1.30	13.6	10.1	15.8	0.82	17.1	14.6
		2S	5-5/12	13.1	1.12	15.2	12.1	16.6	0.31	17.1	16.3
2A	18 mils	1N	5-5/18	20.3	1.03	21.4	19.0	21.6	0.86	23.1	20.7
		1S	5-5/18	20.3	0.86	21.6	19.2	22.1	1.14	23.1	20.3
		2N & 4N	5-5/18	21.0	1.37	23.4	19.5	22.0	0.53	22.7	21.2
		2S	5-5/18	18.8	0.76	20.0	18.1	21.0	1.51	22.9	19.2
2B	18 mils	1N	5-7/18	18.6	0.44	19.2	18.0	21.8	0.58	22.5	21.0
		1S	5-7/18	19.0	2.55	22.9	16.5	20.1	0.93	21.1	18.3
		2N & 4N	5-7/18	19.2	0.82	20.4	18.1	20.8	0.90	21.8	19.3
		2S	5-7/18	19.5	0.64	20.1	18.4	21.5	0.47	22.1	20.8

No. 9 Bundled Bars - $\ell_a = 43"$ (8 readings each side, therefore 16 readings for Valley and Rib)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
3A	12 mils	1N	9-5/12	14.1	1.38	16.3	12.3	16.8	0.94	18.1	15.1
		1S	9-5/12	15.1	1.58	17.4	12.7	18.2	1.40	21.7	16.1
		2N & 4N	9-5/12	15.1	1.66	18.6	12.6	18.2	1.57	21.5	15.7
		2S	9-5/12	13.8	1.82	16.6	10.7	16.5	1.44	18.5	14.1
3B	12 mils	1N	9-12/12	12.9	3.47	18.5	9.0	15.5	2.90	20.7	11.6
		1S	9-12/12	11.7	2.11	14.8	9.0	15.6	2.68	19.8	11.8
		2N & 4N	9-12/12	13.0	2.75	16.8	9.4	15.7	3.06	19.9	11.5
		2S	9-12/12	11.7	2.55	16.7	8.2	15.2	3.10	19.1	10.3
4A	18 mils	1N	9-10/18	18.2	2.01	21.4	15.3	20.7	2.05	23.9	18.2
		1S	9-10/18	20.2	2.35	23.0	16.0	22.8	2.04	25.5	19.5
		2N & 4N	9-10/18	19.9	2.36	23.9	16.7	22.3	2.61	26.1	19.0
		2S	9-10/18	20.2	2.85	24.0	15.7	23.4	2.84	27.5	19.1
4B	18 mils	1N	9-12/18	19.6	2.92	24.4	15.0	22.8	2.35	26.5	19.3
		1S	9-12/18	19.4	2.54	23.6	15.9	22.3	2.08	26.9	19.7
		2N & 4N	9-12/18	20.6	2.24	24.4	17.2	24.0	2.01	26.9	20.8
		2S	9-12/18	20.0	3.30	24.3	15.2	23.1	2.40	27.0	19.5

Table 3.7: Splice-Region Epoxy-Coating Thickness Measurements

No. 5 Bars - $f_y - \ell_a = 14"$ (3 readings each side, therefore 6 readings for Valley and Rib)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
5A	18 mils	1N	5-1/18	20.8	0.68	22.1	20.0	23.2	1.38	24.7	20.8
		1S	5-1/18	21.1	1.02	23.0	20.0	23.6	0.37	24.0	23.0
		2N	5-1/18	20.3	0.68	21.2	19.5	22.3	1.07	23.1	20.2
		2S	5-1/18	21.7	1.43	23.7	20.1	22.3	1.58	24.3	19.8
		3N & 4N	5-1/18	20.6	0.82	21.5	19.4	23.8	0.60	24.6	23.2
		3S	5-1/18	21.3	0.75	22.3	20.2	23.3	1.08	24.5	21.5
5B	18 mils	1N	5-6/18	19.3	1.13	21.2	18.2	22.0	0.66	22.8	21.2
		1S	5-6/18	20.0	1.88	22.8	17.7	21.4	1.04	23.3	20.7
		2N	5-6/18	21.8	1.66	23.6	19.1	21.9	0.59	22.5	21.2
		2S	5-6/18	19.7	1.15	21.4	18.7	21.9	0.77	23.0	21.1
		3N & 4N	5-6/18	21.3	0.77	22.3	20.3	23.0	1.21	24.3	21.1
		3S	5-6/18	21.6	1.57	23.8	19.8	22.4	1.40	24.3	20.1

No. 9 Bars - $f_y - \ell_a = 43"$ (8 readings each side, therefore 16 readings for Valleys and Ribs)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
6A	18 mils	1N	9-3/18	20.7	3.40	26.8	14.9	22.9	2.78	27.3	18.2
		1S	9-3/18	20.7	3.66	26.3	16.4	23.3	2.68	27.6	19.8
		2N	9-3/18	20.3	1.47	22.8	18.3	22.6	1.19	24.6	20.2
		2S	9-3/18	22.4	2.30	25.1	18.6	24.7	2.43	27.8	19.8
		3N & 4N	9-3/18	21.7	2.00	24.3	18.3	24.2	1.63	27.7	21.3
		3S	9-3/18	20.6	3.34	26.0	16.4	23.5	3.25	29.5	18.5
6B	18 mils	1N	9-7/18	21.8	1.81	24.2	18.7	24.4	2.40	27.8	20.3
		1S	9-7/18	20.8	2.23	24.6	17.8	23.6	2.75	27.8	20.1
		2N	9-7/18	21.9	1.71	24.4	18.9	25.2	1.84	28.8	22.8
		2S	9-7/18	21.1	2.67	25.5	16.8	23.7	2.32	26.8	20.0
		3N & 4N	9-7/18	21.6	2.14	24.3	18.4	24.9	2.40	28.3	21.6
		3S	9-7/18	19.7	2.20	22.6	16.4	23.3	2.68	27.6	19.8

No. 5 Bars - $1/2f_y - \ell_a = 10"$ (3 readings each side, therefore 6 readings for Valley and Rib)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
7A	12 mils	1N	5-7/12	13.2	0.64	14.1	12.4	15.4	0.53	16.1	14.5
		1S	5-1/12	13.4	0.37	13.8	12.9	16.3	0.31	16.6	15.9
		2N	5-3/12	13.7	0.15	13.9	13.5	16.7	0.51	17.5	16.2
		2S	5-3/12	13.9	0.90	15.3	12.7	16.3	0.94	18.0	15.3
		3N & 4N	5-1/12	14.4	0.46	15.1	13.9	16.4	0.33	17.0	16.1
		3S	5-1/12	13.3	0.61	14.0	12.3	16.2	0.72	17.0	15.1
7B	12 mils	1N	5-3/12	13.9	0.88	15.0	12.6	16.3	0.80	17.7	15.8
		1S	5-1/12	13.0	0.78	13.7	12.0	15.6	0.38	16.1	15.0
		2N	5-3/12	14.2	1.16	16.0	12.9	17.3	0.67	18.0	16.2
		2S	5-3/12	14.5	0.78	15.5	13.8	16.7	0.58	17.3	15.8
		3N & 4N	5-3/12	13.9	0.88	15.0	12.6	16.3	0.80	17.2	15.0
		3S	5-3/12	13.9	1.08	14.7	12.1	16.5	0.49	17.2	15.7

Table 3.7 (cont.): Splice-Region Epoxy-Coating Thickness Measurements

No. 5 Bars - $1/2f_y - \ell_a = 10"$ (cont.)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
8A	18 mils	1N	5-2/18	20.3	1.66	22.0	17.3	20.8	1.22	22.4	19.2
		1S	5-3/18	22.2	1.20	24.5	21.1	23.5	0.58	24.4	22.9
		2N	5-2/18	23.6	0.89	24.6	22.0	23.9	1.11	25.3	22.5
		2S	5-2/18	22.8	0.98	24.2	21.2	23.4	0.87	24.5	22.0
		3N & 4N	5-3/18	20.3	1.72	22.4	18.0	22.2	0.75	23.0	21.1
		3S	5-2/18	22.7	1.32	24.4	20.4	23.0	1.31	24.3	21.4
8B	18 mils	1N	5-6/18	20.9	1.52	23.0	19.0	21.3	0.85	22.8	20.7
		1S	5-2/18	21.9	0.64	22.4	20.7	23.4	0.81	24.6	22.4
		2N	5-2/18	22.6	0.74	23.9	21.8	22.7	0.75	23.9	21.8
		2S	5-1/18	22.6	1.30	24.5	20.8	23.9	0.83	24.7	22.7
		3N & 4N	5-2/18	21.1	0.77	21.7	19.6	22.2	1.22	23.1	19.8
		3S	5-2/18	21.7	1.13	23.4	20.5	22.6	1.58	23.8	19.8

No. 9 Bars - $1/2f_y - \ell_a = 31"$ (6 readings each side, therefore 12 readings for Valley and Rib)

Specimen	Coating Thickness	Strain Gage	From Bar	Valley				Rib			
				Mean	S.D.	Max	Min	Mean	S.D.	Max	Min
9A	12 mils	1N	9-10/12	13.4	1.12	15.6	12.3	16.8	1.03	18.6	15.4
		1S	9-10/12	14.2	1.56	18.1	12.2	17.9	1.95	21.5	15.1
		2N	9-10/12	15.6	2.00	17.8	12.2	17.5	1.94	19.7	14.9
		2S	9-10/12	14.9	2.47	17.9	11.5	17.0	2.50	20.0	12.7
		3N & 4N	9-10/12	14.5	1.42	16.8	12.0	17.4	1.54	19.0	14.3
		3S	9-10/12	13.8	1.02	15.1	12.2	16.7	1.45	19.0	14.2
9B	12 mils	1N	9-3/12	14.0	1.81	16.8	11.9	16.9	1.83	19.7	14.3
		1S	9-3/12	14.6	2.09	17.7	12.1	17.1	1.95	20.0	13.6
		2N	9-3/12	15.0	1.66	17.3	12.2	17.6	1.65	20.0	15.4
		2S	9-3/12	15.3	1.56	18.5	13.4	17.8	1.75	20.8	15.9
		3N & 4N	9-3/12	14.6	0.87	15.8	13.0	17.5	0.82	18.8	16.1
		3S	9-3/12	14.5	1.81	16.5	11.5	16.9	1.99	19.1	13.6
10A	18 mils	1N	9-5/18	19.4	2.13	22.6	16.7	22.1	2.51	25.2	18.5
		1S	9-5/18	18.5	1.42	21.2	16.8	21.1	1.64	23.4	18.8
		2N	9-5/18	19.9	2.09	23.7	17.2	22.0	1.83	24.1	18.9
		2S	9-5/18	19.6	2.50	23.9	16.1	21.5	2.38	24.7	16.7
		3N & 4N	9-5/18	19.1	2.68	23.1	15.8	21.7	2.51	26.0	18.0
		3S	9-5/18	19.5	1.60	22.0	17.2	22.1	1.77	24.7	20.1
10B	18 mils	1N	9-15/18	18.4	2.05	22.8	16.4	21.2	1.98	25.7	19.3
		1S	9-15/18	19.0	1.69	21.6	16.5	22.2	2.23	25.0	19.5
		2N	9-15/18	19.7	1.08	21.3	18.1	22.7	1.68	24.7	20.1
		2S	9-15/18	19.8	1.17	21.8	18.5	22.9	1.81	26.1	20.7
		3N & 4N	9-15/18	17.9	1.52	22.0	15.9	20.6	1.66	25.1	18.4
		3S	9-15/18	19.6	0.95	21.1	18.3	22.5	1.36	25.2	20.9

Table 3.7 (cont.): Splice-Region Epoxy-Coating Thickness Measurements

LVDT	Distance from Center (in.)	Model Number	Range (in.)
1N	83	DC-EC 2000	± 2.0
2N	51	DC-D 1000	± 1.0
5N	30	DC-E 500	± 0.500
3N	15	DC-E 250	± 0.250
4N	0	DC-E 500	± 0.500
3S	-15	DC-E 250	± 0.250
5S	-30	DC-E 500	± 0.500
2S	-51	DC-EC 1000	± 1.0
1S	-83	DC-E 2000	± 2.0

Table 3.8: LVDT Summary

Load Cell	Location	Model	Capacity	Serial Number
1	North-East	Lebow 3156-50k	50 kips	2436
2	North-West	Lebow 3156-150k	150 kips	3100
3	South-East	Lebow 3156-150k	150 kips	2362
4	South-West	Tokyo Sokki KCB-500kNA	500 kN	ALU03005

Table 3.9: Load Cell Summary

Specimen: 1A (No. 5 Bundled Bars, 12 mils, $\ell_a = 14"$)

h =		12-1/8"							
d =		9.300"							
b _{bottom} =		20-1/8"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	14-3/16"	2-1/4"	2-1/4"	2-3/16"	2-5/16"	2-1/16"	2-1/16"	2-1/8"	2-1/4"
East	14-1/16"	2-1/4"	2-1/4"	2-1/4"	2-3/16"	2-1/16"	1-15/16"	1-3/4"	2-1/4"

Specimen: 1B (No. 5 Bundled Bars, 12 mils, $\ell_a = 14"$)

h =		12-1/8"							
d =		9.268"							
b_{bottom} =		20-1/4"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	14"	2-1/4"	2-1/4"	2-3/16"	2-5/16"	1-7/8"	1-7/8"	1-15/16"	2-3/16"
East	14-1/16"	2-3/16"	2-1/4"	2-3/8"	2-5/16"	2-3/16"	2-1/16"	2"	2-5/16"

Specimen: 2A (No. 5 Bundled Bars, 18 mils, $\ell_a = 14"$)

h =		12-3/16"							
d =		9.315"							
b _{bottom} =		20"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	14-1/16"	2-1/8"	2-1/16"	2-1/16"	2-5/16"	1-7/8"	1-7/8"	2"	2-3/16"
East	14-1/16"	2-1/16"	2-1/8"	2-1/8"	2-7/16"	2-3/8"	2-1/4"	2-1/16"	2-3/8"

Specimen: 2B (No. 5 Bundled Bars, 18 mils, $\ell_a = 14"$)

h =		12-1/8"							
d =		9.175"							
b _{bottom} =		20-1/8"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	14"	2-3/16"	2-1/8"	2-1/16"	2-1/4"	1-7/8"	1-15/16"	1-15/16"	2-1/8"
East	14-1/16"	2"	2-1/8"	2-1/4"	2-3/8"	2-3/16"	2-1/8"	2-1/16"	2-7/16"

Specimen: 3A (No. 9 Bundled Bars, 12 mils, $\ell_a = 43"$)

h =		12-3/16"							
d =		8.523"							
b_{bottom} =		20-1/16"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	43-1/8"	2-1/4"	2-1/8"	1-7/8"	2-11/16"	2"	2-5/16"	2-5/8"	2-11/16"
East	43"	2-1/16"	2-5/16"	2-9/16"	2-5/8"	2-9/16"	2-5/16"	2-1/16"	2-3/4"

Specimen: 3B (No. 9 Bundled Bars, 12 mils, $\ell_a = 43"$)

h =		12-1/8"							
d =		8.398"							
b _{bottom} =		20-1/4"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	43-3/16"	2-5/16"	2-5/16"	2-3/16"	2-7/8"	2-3/16"	2-7/16"	2-5/8"	2-11/16"
East	42-15/16"	1-15/16"	2-1/8"	2-3/8"	2-11/16"	2-11/16"	2-3/8"	2-3/16"	2-13/16"

Table 3.10: As-Built Data for All Specimens

Specimen: 4A (No. 9 Bundled Bars, 18 mils, $\ell_a = 43"$)

h =		12-1/8"							
d =		8.523"							
b _{bottom} =		20"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	43-3/16"	2-1/4"	2"	1-13/16"	2-11/16"	1-7/8"	2-1/8"	2-1/2"	2-9/16"
East	43-3/16"	2-1/8"	2-1/4"	2-1/2"	2-7/16"	2-5/16"	2-1/16"	1-7/8"	2-13/16"

Specimen: 4B (No. 9 Bundled Bars, 18 mils, $\ell_a = 43"$)

h =		12-1/8"							
d =		8.398"							
b_{bottom} =		20-1/8"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	42-7/8"	2-3/8"	2-3/16"	2"	2-7/8"	2"	2-1/8"	2-7/16"	2-1/2"
East	43-1/16"	1-13/16"	2-1/16"	2-7/16"	2-11/16"	2-9/16"	2-5/16"	2"	2-13/16"

Specimen: 5A (No. 5 Bars, 18 mils, $\ell_a = 14"$)

h =		12-1/8"							
d =		9.75"							
b _{bottom} =		20-1/8"							
Splice Location	ℓ _a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	14-1/16"	2-1/16"	2-1/16"	2-1/16"	2-1/4"	2-3/16"	2-3/16"	2-1/4"	2-1/4"
Center-W	14-1/8"	---	6-1/16"	---	2-3/16"	2-1/16"	2-1/16"	2-3/16"	2-3/16"
Center-E		---	6-1/16"	---					
East	14-3/16"	2-1/8"	2-1/8"	2-3/16"	2-3/16"	2-1/8"	2-1/8"	2-1/8"	2-3/16"

Specimen: 5B (No. 5 Bars, 18 mils, $\ell_a = 14"$)

h =		12-1/8"							
d =		9.73"							
b_{bottom} =		20-1/4"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	14-3/16"	2-1/8"	2-3/16"	2-1/8"	2-3/16"	2-1/8"	2-1/8"	2-1/8"	2-3/16"
Center-W	14-1/8"	---	6"	---	2-1/8"	2-1/8"	2-1/16"	2-1/16"	2-1/16"
Center-E		---	6"	---			2-1/16"	2-1/16"	2-1/16"
East	14-1/16"	2-1/8"	2-3/16"	2-3/8"	2-1/8"	2-1/8"	2-1/16"	2-3/16"	2-3/16"

Specimen: 6A (No. 9 Bars, 18 mils, $\ell_a = 43"$)

h =		12-1/8"							
d =		9.436"							
b_{bottom} =		20"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	43"	2"	2"	2"	2-1/8"	2-3/16"	2-1/16"	2-3/16"	2-3/16"
Center-W	43-1/16"	---	4-5/16"	---	2-1/4"	2-3/16"	2-3/16"	2-1/8"	2-1/8"
Center-E		---	4-5/16"	---					
East	43-5/16"	2-1/8"	2-1/8"	2-1/8"	2-3/16"	2-1/8"	2-1/8"	2-1/8"	2-1/8"

Table 3.10 (cont.): As-Built Data for All Specimens

Specimen: 6B (No. 9 Bars, 18 mils, $\ell_a = 43"$)

h =		12-1/8"							
d =		9.29"							
b_{bottom} =		20-1/8"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	43-5/16"	2-1/8"	2-3/16"	2-1/4"	2-3/8"	2-5/16"	2-5/16"	2-5/16"	2-5/16"
Center-W	43-3/8"	---	4-3/16"	---	2-3/8"	2-5/16"	2-1/4"	2-1/4"	2-1/4"
Center-E		---	4-3/8"	---					
East	43-5/8"	2-1/16"	2-1/8"	2-1/4"	2-1/4"	2-3/16"	2-1/4"	2-1/4"	2-1/4"

Specimen: 7A (No. 5 Bars, 12 mils, $\ell_a = 10"$)

h =		12-3/16"							
d =		9.719"							
b_{bottom} =		20-1/8"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	10-3/16"	2-1/16"	2-1/8"	2-1/8"	2-1/4"	2-1/8"	---	2-3/16"	2-1/4"
Center-W	10-1/8"	6-1/16"	---	6-1/16"	2-5/16"	2-3/16"	---	2-3/16"	2-5/16"
Center-E		5-7/8"	---	5-7/8"					
East	10-1/8"	2"	2"	2"	2-5/16"	2-1/4"	---	2-1/4"	2-3/16"

Specimen: 7B (No. 5 Bars, 12 mils, $\ell_a = 10"$)

h =		12-3/16"							
d =		9.813"							
b_{bottom} =		20-5/16"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	10-1/16"	2-3/16"	2-1/4"	2-1/4"	2-3/16"	2-1/8"	---	2"	2-3/16"
Center-W	10-1/16"	5-15/16"	---	5-15/16"	2-3/16"	2-1/8"	---	2-1/16"	2-1/4"
Center-E	---	5-15/16"	---	5-15/16"	---	---	---	---	---
East	10-1/16"	2-5/16"	2-1/4"	2-1/4"	2-3/16"	2-1/16"	---	2"	2-3/16"

Specimen: 8A (No. 5 Bars, 18 mils, $\ell_a = 10"$)

h =		12-1/8"							
d =		9.375"							
b_{bottom} =		20"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	10-1/16"	2-1/16"	2-1/16"	2-1/16"	2-1/8"	1-15/16"	---	2-1/8"	2-1/4"
Center-W	10-3/16"	5-15/16"	---	5-15/16"	2-1/4"	2-1/4"	---	2-7/16"	2-5/16"
Center-E		5-15/16"	---	5-15/16"					
East	10-5/16"	2-1/16"	2-1/16"	2-1/16"	2-1/4"	2-1/4"	---	2-3/8"	2-5/16"

Specimen: 8B (No. 5 Bars, 18 mils, $\ell_a = 10"$)

h =		12-1/8"							
d =		9.688"							
b_{bottom} =		20-1/16"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	10-1/16"	2-1/8"	2-1/16"	2-1/16"	2-3/8"	2-3/16"	---	2-1/8"	2-3/16"
Center-W	10"	6-1/16"	---	6-1/16"	2-5/16"	2-1/8"	---	2-1/4"	2-1/4"
Center-E		5-7/8"	---	5-15/16"					
East	10"	2-1/8"	2-1/8"	2-1/8"	2-1/4"	2-1/8"	---	2-1/8"	2-1/4"

Table 3.10(cont.): As-Built Data for All Specimens

Specimen: 9A (No. 9 Bars, 12 mils, $\ell_a = 31"$)

h =		12-1/4"							
d =		9.249"							
b_{bottom} =		20-1/8"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	31-1/4"	2"	2-1/16"	2"	2-3/8"	2-3/8"	---	2-7/16"	2-3/8"
Center-W	31-1/4"	4-5/16"	---	4-5/16"	2-3/8"	2-3/8"	---	2-1/2"	2-7/16"
Center-E		4-5/16"	---	4-5/16"					
East	31-1/4"	2"	1-15/16"	2"	2-3/8"	2-7/16"	---	2-1/2"	2-3/8"

Specimen: 9B (No. 9 Bars, 12 mils, $\ell_a = 31"$)

h =		12"							
d =		9.311"							
b _{bottom} =		20-1/4"							
Splice Location	ℓ _a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	31-1/16"	2-1/4"	2-1/4"	2-1/8"	2-1/16"	2-1/16"	---	2-1/16"	2-1/8"
Center-W	31-1/16"	4-3/8"	---	4-7/16"	2-1/8"	2-1/8"	---	2-3/16"	2-3/16"
Center-E		4-3/8"	---	4-3/8"					
East	31-1/16"	2-1/8"	2-1/16"	2-3/16"	2-1/8"	2-1/8"	---	2-3/16"	2-1/8"

Specimen: 10A (No. 9 Bars, 18 mils, $\ell_a = 31"$)

h =		12-1/8"							
d =		9.186"							
b _{bottom} =		20-1/16"							
Splice Location	ℓ _a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	31-1/8"	2-1/16"	2-1/16"	2-1/16"	2-3/8"	2-3/8"	---	2-3/8"	2-5/16"
Center-W	31-3/16"	4-5/16"	---	4-3/8"	2-7/16"	2-3/8"	---	2-3/8"	2-3/8"
Center-E		4-5/16"	---	4-5/16"					
East	31-5/16"	2-1/16"	2-1/16"	2-1/16"	2-5/16"	2-3/8"	---	2-3/8"	2-5/16"

Specimen: 10B (No. 9 Bars, 18 mils, $\ell_a = 31"$)

h =		12-1/8"							
d =		9.29"							
b_{bottom} =		20-1/16"							
Splice Location	ℓ_a	Side Cover			Top Cover				
		S-C	C	N-C	S	S-C	C	N-C	N
West	31-3/16"	2"	2"	2"	2-3/16"	2-5/16"	---	2-1/4"	2-1/4"
Center-W	31-1/4"	4-5/16"	---	4-5/16"	2-3/16"	2-5/16"	---	2-1/4"	2-1/4"
Center-E		4-5/16"	---	4-5/16"					
East	31-1/4"	2"	2-1/8"	2-1/8"	2-3/16"	2-5/16"	---	2-3/16"	2-1/4"

Table 3.10 (cont.): As-Built Data for All Specimens

Specimen No.	Test Date	Equipment for Test Setup Load	Test Setup Load (lb)
1B	6/16/04 - 6/18/04	1-Load Cell, 1-Red Ram, Load Roller (Figure 3.10)	189.5
2A	6/24/04 - 8/4/04	1-Load Cell, Load Roller (Figure 3.11)	133.5
1A	08/31/2004		
2B	09/09/2004		
3A	10/7/04 - 12/3/04	2-Steel Plate, 2-Load Cell, 2-Center-Hole Ram, Blue Steel Beam, Load Roller (Figure 3.12)	848
4A	12/10/2004		
4B	12/21/2004		
3B	01/24/2005		
5A	02/09/2005		
5B	02/17/2005		
6A	03/03/2005		
6B	03/08/2005		
9A	09/28/2005		
9B	10/28/2005		
10A	11/10/2005		
10B	12/21/2005		
7A	04/30/2006		
8A	05/12/2006		
8B	05/19/2006		
7B	05/30/2006		

Table 3.11: Test Setup Load (self-weight of concrete beam not included)

(a) Cast 1

(b) Cast 2

Figure 3.1: Strength Gain for all Casts

(c) Cast 3

(d) Cast 4

Figure 3.1 (cont.): Strength Gain for all Casts

(e) Cast 5

Figure 3.1 (cont.): Strength Gain for all Casts

(a) No. 5 Bars

(b) No. 9 Bars

Figure 3.2: Stress vs. Strain (strain gage) curves for steel reinforcing bars: (a) No. 5 Bars and (b) No. 9 Bars

Figure 3.3: Typical Strain Gage and LVDT Layout for (a) Bundled Bar and (b) Single Splice Specimens (top views of beam)

Figure 3.4: Typical Cross-Section in the Splice-Region for (a) Bundled Bar and (b) Single Splice Specimens

Figure 3.5: Reinforcement Cage

Figure 3.6: Formwork with reinforcement cage before cast

Figure 3.7: Cast Setup

Figure 3.8: Concrete cast directly into formwork from chute

Figure 3.9: Test Setup (Shear and Moment diagrams show the effect of only the point loads, including weight of test set-up.)

Figure 3.10: Specimen 1B Loading Point Test Setup at Failure

Figure 3.11: Specimen 1A, Specimen 2A and Specimen 2B Loading Point Test Setup at Failure

Figure 3.12: Specimen 3A through Specimen 10B Loading Point Test Setup at Failure

4 PRESENTATION OF MEASURED TEST DATA

4.1 Introduction

This chapter presents the following test data for all specimens in the experimental program described in Chapter 3: (i) failure load, (ii) strain gage data, (iii) deflection data, (iv) crack patterns, (v) crack widths, and (vi) description of load events.

4.2 Failure Load

Two definitions of load levels are used in this chapter: Failure Load and Applied Load. These loads refer to the point load applied at each end of the specimen. The Failure Load is the highest load the beam resisted at which the first splice failure occurred. This load level corresponded to the maximum load carried by the specimen in all cases. The Failure Load is equal to the maximum Applied Load for a given specimen plus the self-weight of the loading point test setup and the effect of the specimen self-weight. The effect of the specimen self-weight was calculated using the largest self-weight moment at the end of the splice. On the other hand, the Applied Load does not include the self-weight of the loading point test setup or the effect due to the self-weight of the concrete beam, except in Specimen 3A through 10B which included 1 kip (1 kip = 1000 lb) to approximate the self-weight of the loading point test setup. Also the Applied Load term is used to refer to any load level and is identified as needed in the report.

The weight of the loading point test setup for each specimen is presented in Table 3.11 and discussed in Section 3.6. Table 4.1 is a summary of the Applied Load at Failure and the Failure Load, as previously defined, for each specimen. It should be noted that there was no sudden splice failure in Specimen 5B, as in there was no sudden decrease of Applied Load due to a splice failure, but there were significant splitting cracks present when the test was deemed to be completed. This test was terminated when the beam reached a point where the deflection continued to increase with no increase in Applied Load as it will be discussed in Section 4.4. This is also discussed in more detail with respect to observed crack patterns in Section 4.5.

4.3 Strain Gage Data

A summary of the strain gage data for each beam is listed in Table 4.2. The first column in the table indicates the specimen tested. Failure Strain refers to the largest strain recorded, just before failure, for a given strain gage. Figure 3.3 shows the typical layout of strain gages for the specimens tested. The gage locations 1 through 3 in that figure show the placement of the strain gages on the bars as discussed in Section 3.4.1. The values under the heading South Failure Strain and North Failure Strain are the averages of the South (S) and North (N) failure strains for each strain gages listed under the table column heading Failed Splice Location. The reinforcement strain gages showed strains up to 7000 $\mu\epsilon$. This value is on the yield plateau of the Stress vs. Strain curves for the reinforcement used (Figure 3.2). After 7000 $\mu\epsilon$, the strain gages would get to a point

where they no longer worked, therefore a note of Past Yield strain under the headings South Failure Strain, North Failure Strain, or Average Failure Strain means that the strain in the reinforcement had reach past the point that the strain gage worked. It was inferred from load readings that the steel reinforcement had reached the strain hardening region of the Stress vs. Strain curves. The values under the table heading Average Failure Strain are the average of the South Failure Strain and North Failure Strain values for each specimen tested.

The first failure was defined as a splice failure at the highest load. This is also the definition of Failed Splice Location. The proceeding additional events in the same specimen were at lower loads when either a different splice failed or flexural failure (concrete crushing in the compression zone) of the beam started to occur. Failed Splice Location is noted by **West**, **East**, or all the splices (**All**) (also see Figure 3.3 and Table 3.10).

The first splice that failed was determined by observation of the crack pattern and/or the strain gage data at the first failure. If a certain splice was not determined to be the failed splice due to either no clear sign of which splice failed first or all the splices in the beam failed at the same time, all the splices in the specimen were considered to be the failed splice as noted by **All** under the heading Failed Splice Location. Listed in the parentheses beside the failed splice are the strain gages, notation corresponding to Figure 3.3, that were used to report the Failure Strain. For example, Specimen 7B has “All (1S - 3S, 1N & 2N)” stated in the Failed Splice Location, which means all splices failed at the same time (“All”), South Failure Strain is the average of the failure strains for strain gages 1S, 2S, and 3S, and North Failure Strain is the average of the failure strains for strain gages 1N and 2N. Under the table heading Splice Length is the length of the splice for the Failed Splice Location in each specimen, which is the same as the highlighted Splice Location in the As-Built Data for All Specimens (Table 3.10). In the specimens where the Failed Splice Location is identified as **All** in the table, the shortest splice length is reported.

4.4 Deflection Data

A summary of the deflection data for each beam is listed in Table 4.3. In this table, the deflections listed were measured at the Failure Load for each beam using the acquired LVDT data. The LVDT Locations refer to the notation used in Figure 3.3 and Table 3.8. The sign convention for the deflection measurements is a positive deflection is a downward displacement and a negative deflection is an upward displacement for the profile of the beam. The deflection measurements at LVDT locations 5S and 5N (see Figure 3.3) recorded deflection at the supports; therefore deflection measurements listed in this table have not been modified at this point to account for support settlement. It was noted before that there was no brittle splice failure in Specimen 5B, but there were significant splitting cracks present when the test was terminated. This test was determined to be finished when the beam reached a point where deflections continue to increase without significant increases in the load. In comparing Specimens 5A and 5B in Table 4.3, the end of the test for Specimen 5B was when the increase in deflection was

about 0.14 in. at the center of the beam (LVDT 4N) and 0.5 in. at the beam ends (LVDTs 1S and 1N) past the measured maximum deflections observed for the other similarly constructed Specimen 5A.

4.5 Crack Patterns

Two different groups of crack map figures are discussed in this section. The first group consists of the test region crack map figures which are top views in groups of four (Figures Figure 4.1 through Figure 4.5). The second group of crack map figures consist of a figure for each specimen that shows a photo of the side view of the specimen and a top view drawing with the locations of the strain gages on the reinforcing bars and the concrete side relative to the cracks (Figures Figure 4.6 through Figure 4.25).

The top views of the test region crack map shown in Figure 4.1 through Figure 4.5 include the two support locations (Support) and the splice-region shown as a shaded gray region. The longitudinal dashed lines in the splice-region represent the location of the side clear cover for West and East splices and the centerline of the Center splice (notations consistent with Figure 3.3 and Table 3.10). The point at which the longitudinal dashed line tees into the perpendicular dashed line at the border of the splice region represents the end of the splice; therefore the distance between the short vertical dashed lines is the length of the splice. The circled letters, **A** and **B**, identify specific cracks, which are the same cracks identified in each crack map figure for each specimen as shown in Figure 4.6 through Figure 4.25. This crack notation helps link all the crack map figures and pictures together and identifies the specific crack used for crack width measurements discussed in Section 4.7. On the top view crack map for each specimen, there are two different sets of numbers that represent the Applied Load (kips) at which each crack was first observed. The first set of numbers aligned at the bottom of each beam figure represent the Applied Load at which the adjacent flexural crack (vertical crack) was first observed. The numbers over the longitudinal top cracks (splitting cracks), which are within the concrete specimen, represent the Applied Load (kips) at which each adjacent splitting crack was observed and the location of the end of the crack corresponding to that Applied Load. The overlapping **CL** at the bottom of each specimen identifies the centerline of the splice. The thinner crack lines with no numbers associated with them represent the cracks that occurred after the highest Applied Load (Failure Load) had been reached due to continued loading causing another splice failure to occur.

Figure 4.6(a) through Figure 4.25(a) show a photo of the side view of Specimen 1A through Specimen 10B. The Side View presented in these figures is of the East side of each beam showing the crack progression for the duration of the test. The vertical cracks are flexural cracks, and the horizontal cracks are splitting cracks. The numbers represent the Applied Load (kips) at which each adjacent crack was formed and the location of the end of the crack at that load. The number closest to the top of the beam for a given flexural crack is the Applied Load (kips) at which the crack first formed. The supports are shown near the ends of each photo where there is a roller in-between two rectangular plates. The center of the roller is the Support section shown in the Top View figures.

The drawing in Figure 4.6(b) through Figure 4.25(b) show the location of the strain gages in relation to the observed cracks (Top View figures). The numbers aligned in the vertical direction at the bottom of each Top View figure correspond to the Applied Load (kips) at which the flexural crack was first noted. The Top View in Figure 4.6(b) through Figure 4.25(b) is the same figure shown in Figure 4.1 through Figure 4.5, except for the following: the strain gage locations and dimensions away from closest crack are shown, the dashed horizontal lines are the center line of the steel reinforcing bars forming the splice, and the support line is at the section cut at the South and North ends.

It was noted before that there was no true splice failure for Specimen 5B, but there were significant splitting cracks present when the test was determined to be done. Figure 4.26 shows a photo of these splitting cracks, the horizontal cracks on the top and the side of the beam. This view is of the West side of the beam and crack marked with the circled letters, **A** and **B**, are consistent with all the other crack map figures.

4.6 Crack Spacing

The average flexural crack spacing in the constant moment region is presented in Table 4.4 for each specimen identified under heading Specimen No. in the table. Note that the flexural cracks that occurred prior to failure, and not at failure, are the ones that are used to determine the flexural crack spacing. The First and Last flexural crack in the constant moment region (between the supports) are the first crack in the constant moment region closest to the support that is not the first crack appearing (at the smallest Applied Load) at the support line. The values under the heading Number of Spaces between Cracks are the number of spaces between the First and Last flexural crack. The distance from the First and Last crack is measured and is referred to as the Distance between the First and Last Flexural Crack and values under this table heading. The Average Test Flexural Crack Spacing is determined by dividing the Distance between the First and Last Flexural Crack by the Number of Spaces between Cracks and the values are given in Table 4.4.

4.7 Crack Widths

Crack widths of selected flexural cracks were taken during the test prior to failure, which is summarized in Table 4.5. The values under the heading Coating Thickness are the minimum epoxy-coating thickness for the range specified (12 mils for the 12-15 mils range, and 18 mils for the 18-21 mils range) so measurements can be compared within a Design Group based on their coating thickness, as further discussed in Chapter 5 (Section 5.6). Under the heading Design Group, the bar size used in the specimens (No. 5 bars or No. 9 bars), bar configuration (bundled or single splices if unspecified), and the design bar stress at failure ($1/2 f_y$ or f_y) are identified. Column Heading **A** and **B** under the major heading Average Crack Width represent crack locations noted in the crack map figures and pictures (Figure 4.1 through Figure 4.26). The Average Crack Width values for each selected crack under the headings **A** and **B** represent an average of three measurements either taken, for Single Splice beams, above each reinforcing bar or taken, for Bundled Bar beams, above each of the two reinforcing bundles and one more measurement in the

center of the Bundled Bar beam. The values under the heading Applied Load in Table 4.5, corresponding to a given Average Crack Width measurement for a given specimen, are the Applied Load level at which the measurement was taken. The values under the heading Failure Load are the failure loads for each specimen as defined in Section 4.2. There is as significant difference between the Failure load level and the maximum Applied Load corresponding to the crack width measurement for each specimen. This is due to safety concerns associate with the extremely brittle mode of failure associated with these tests. Also note, that for one of the specimens in each group, as defined under the heading Design Group, the measurements were stopped at an Applied Load level lower than the other specimens in the same group. This is because before the first Failure Load was known for that group, measurements were stopped before the Nominal Calculated (Nom. Calc.) Failure Load was reached.

The values under the heading Nominal Calculated (Nom. Calc.) Failure Load were estimated based on nominal design properties presented and discussed in Section 3.2.2. After the first test in each Design Group was performed, the measurements in the other specimens in the same group were taken from 65% to 95% of the Failure Load of the first beam tested in that group. For instance, in the first Design Group shown in Table 4.5 (No. 5 bundle, $1/2 f_y$), the first specimen tested was 1B, and only one average measurement is shown under the heading **A** and **B** for the Applied Load level of 12 kips. The other three specimens in that Design Group have Average Crack Width Measurements for two load levels shown: a measurement at 12 kips to compare with Specimen 1B and one at 17.5 kips corresponding to a load level of 75% of the Failure Load of Specimen 1B. Further when two Design Groups had similarities, for instance the first two design groups in Table 4.5 (No. 5 bundle, $1/2 f_y$ and No. 9 bundle, $1/2 f_y$), the measurements on the specimens belonging to the second group tested were taken also at two Applied Load levels using the experience gained in the first specimen tested of the first Design Group tested.

The whole point of this exercise was, with safety in mind, to get closer to the failure loads with the crack width measurements. The first beams tested in each Design Group used as a reference to estimate the maximum Applied Load at which these measurements were taken for other specimens in the same or similar Design Group were 1B and 7A. It should be noted that this data was acquired with the same procedure and same operator for every test; therefore the relative crack widths are a good comparison, which will be presented in Chapter 5 (Section 5.6).

4.8 Load Events

There are three significant events identified during a given test: first flexural crack, first splitting crack, and failure of the beam. The First Flexural Crack is when the concrete starts to depend on the reinforcement to provide the strength in tension in the tensile zone by engaging the steel, while the concrete contributes more in the compression zone. The First Splitting Crack is the first sign that a splice failure will occur and this was also discussed in Section 4.5. Finally splice failure was noted. Each of these events has an associated Applied Load corresponding to the event. These significant loading events are

presented in Table 4.6. Failure Load in the table is the Failure Load defined in Section 4.2.

4.9 Summary

The test data for all specimens presented in this chapter include: (i) failure loads, (ii) strain gage data, (iii) deflection data, (iv) crack patterns, (v) crack widths, and (vi) description of load events. These data is analyzed in Chapter 5 and salient conclusions are stated in Chapter 6 including design implications.

Specimen No.	Applied Load at Failure (kips)	Failure Load (kips)
1A	23.5	24.1
1B	22.5	23.2
2A	23.7	24.4
2B	23.0	23.6
3A	53.6	54.1
3B	55.7	56.1
4A	54.6	55.1
4B	54.6	55.0
5A	17.1	17.5
5B	17.7	18.0
6A	42.9	43.4
6B	39.9	40.3
7A	12.3	12.7
7B	12.6	12.9
8A	12.5	12.8
8B	13.2	13.5
9A	33.1	33.5
9B	33.8	34.2
10A	34.7	35.1
10B	33.2	33.6

Table 4.1: Failure Load Summary

Specimen No.	South Failure Strain (μ strain)	North Failure Strain (μ strain)	Average Failure Strain (μ strain)	Failed Splice Location	Splice Length
1A	2728	6950	4839	West (2S & 1N)	14-3/16"
1B	1604	2440	2022	East (1S & 1N)	14-1/16"
2A	2309	4915	3612	All (1S, 2S, 1N & 2N)	14-1/16"
2B	2546	2369	2458	East (1S & 1N)	14-1/16"
3A	2330	1860	2095	East (1S & 1N)	43"
3B	2306	1951	2129	East (1S & 1N)	42-15/16"
4A	2047	2355	2201	West (2S & 1N)	43-3/16"
4B	2268	2008	2138	East (1S & 1N)	43-1/16"
5A	Past Yield	6057	6057	West (3S & 3N)	14-1/16"
5B	Past Yield	Past Yield	Past Yield	All (1S - 3S & 1N - 3N)	14-1/16"
6A	3056	3148	3102	All (1S, 2S, 2N & 3N)	43"
6B	2628	2650	2639	All (1S - 3S & 1N - 3N)	43-5/8"
7A	1557	2239	1898	West (3S & 3N)	10-3/16"
7B	2141	2091	2116	All (1S - 3S, 1N & 2N)	10-1/16"
8A	1847	2454	2151	East (1S & 1N)	10-5/16"
8B	2781	2141	2461	East (1S & 1N)	10"
9A	1859	2108	1984	All (1S - 3S & 1N - 3N)	31-1/4"
9B	2200	2211	2206	All (1S - 3S & 1N - 3N)	31-1/16"
10A	2337	2155	2246	All (1S, 3S & 1N - 3N)	31-1/8"
10B	2421	2067	2244	All (1S & 1N - 3N)	31-3/16"

Table 4.2: Strain Gage Data Summary

Specimen No.	Deflection at LVDT Location (in.)								
	1S	2S	5S	3S	4N	3N	5N	2N	1N
1A	1.072	0.398	---	-0.137	-0.175	-0.127	---	0.393	1.095
1B	0.945	0.362	---	-0.103	-0.148	-0.047	---	0.327	0.928
2A	0.960	0.333	---	-0.125	-0.153	-0.116	---	0.358	0.964
2B	1.078	0.410	---	-0.11*	-0.165	-0.11*	---	0.401	1.085
3A	1.336	0.509	0.005	-0.143	-0.192	-0.136	0.029	0.502	1.405
3B	1.451	0.544	0.022	-0.149	-0.200	-0.143	0.035	0.530	1.449
4A	1.351	0.517	0.004	-0.123	-0.170	-0.122	0.035	0.507	1.383
4B	1.370	0.514	0.039	-0.120	-0.161	-0.103	0.068	0.532	1.419
5A	2.113	0.853	0.059	-0.286	-0.391	-0.305	0.014	0.748	2.039
5B	2.596	0.997	0.008	---	-0.534	---	0.014	1.000	2.620
6A	1.167	0.428	0.026	-0.100	-0.128	-0.096	0.035	0.426	1.157
6B	1.074	0.404	0.034	-0.082	-0.107	-0.072	0.049	0.409	1.099
7A	0.743	0.296	0.033	-0.067	-0.116	-0.088	0.010	0.273	0.734
7B	0.692	0.273	0.019	-0.081	-0.118	-0.078	0.024	0.281	0.726
8A	0.740	0.299	0.025	-0.079	-0.125	-0.098	0.007	0.299	0.800
8B	0.784	0.304	0.019	-0.088	-0.110	-0.073	0.036	0.319	0.831
9A	0.894	0.333	0.025	-0.085	-0.113	-0.088	0.021	0.334	0.931
9B	0.910	0.329	0.022	-0.089	-0.115	-0.088	0.028	0.354	0.972
10A	0.951	0.354	0.030	-0.087	-0.112	-0.085	0.033	0.352	0.983
10B	0.897	0.332	0.028	-0.073	-0.094	-0.060	0.065	0.379	0.979

*Estimate since LVDT went offline a few data points before this load's deflection was taken.

Note: "---" for either no deflection data available or LVDT went offline several data points before recording this load's deflection.

Table 4.3: Deflection Data Summary

Specimen No.	Number of Spaces between Cracks	Distance between First and Last Flexural Crack (in.)	Average Test Flexural Crack Spacing (in.)
1A	6	43.2	7.2
1B	6	43.5	7.2
2A	6	43.5	7.3
2B	6	39.4	6.6
3A	5	40.6	8.1
3B	5	40.9	8.2
4A	6	42.3	7.1
4B	6	41.9	7.0
5A	5	38.5	7.7
5B	5	40.8	8.2
6A	6	41.1	6.9
6B	7	41.2	5.9
7A	4	35.2	8.8
7B	4	27.0	6.8
8A	4	35.7	8.9
8B	3	30.6	10.2
9A	6	47.3	7.9
9B	6	42.5	7.1
10A	6	46.2	7.7
10B	6	43.4	7.2

Table 4.4: Average Flexural Crack Spacing in Constant Moment Region Prior to Failure

Specimen No.	Average Crack Width (in.)		Applied Load (kips)	Failure Load (kips)	Coating Thickness	Design Group	Nom. Calc. Failure Load (kips)
	A	B					
1A	0.016	0.016	17.5	24.1	12 mils	No. 5 bundle*, $1/2 f_y$	13.7
	0.012	0.012	12				
1B	0.011	0.011	12	23.2			
2A	0.018	0.020	17.5	24.4	18 mils		
	0.011	0.011	12				
2B	0.021	0.021	17.5	23.6			
	0.016	0.015	12				
3A	0.021	0.021	32	54.1	12 mils	No. 9 bundle*, $1/2 f_y$	30.2
	0.016	0.015	28				
3B	0.022	0.024	36	56.1			
	0.015	0.016	28				
4A	0.015	0.019	36	55.1	18 mils		
	0.006	0.011	26				
4B	0.017	0.015	40	55.0			
	0.014	0.010	28				
5A	0.039	0.039	14	17.5	18 mils	No. 5, f_y	8.8
	0.009	0.007	7				
5B	0.038	0.019	13	18.0			
	0.010	0.009	7				
6A	0.013	0.019	26	43.4	18 mils	No. 5, f_y	26.6
6B	0.019	0.018	26	40.3			
7A	0.017	0.013	8.5	12.7	12 mils	No. 5, $1/2 f_y$	8.6
7B	0.022	0.021	12	12.9			
	0.011	0.009	8		12.8		
8A	0.024	0.028	11	13.5			
	0.013	0.015	7.5				
8B	0.025	0.028	12				
	0.015	0.015	8				
9A	0.029	0.014	30	33.5	12 mils	No. 9, $1/2 f_y$	25.3
	0.018	0.010	22				
9B	0.020	0.018	26	34.2			
	0.016	0.014	20				
10A	0.019	0.021	28	35.1	18 mils		
	0.014	0.013	20				
10B	0.017	0.011	28	33.6			
	0.013	0.005	20				

*Bundle is a three-bar bundle

Table 4.5: Average Flexural Crack Widths Prior to Failure

Specimen No.	First Flexural Crack Load (kips)	First Splitting Crack Load (kips)	Failure Load (kips)
1A	4.5	12	24.1
1B	6	18	23.2
2A	4.5	12.5	24.4
2B	4.5	16	23.6
3A	3	24	54.1
3B	3	24	56.1
4A	3	34	55.1
4B	3	22	55.0
5A	2	12	17.5
5B	4.5	16.5	18.0
6A	3.5	10	43.4
6B	5	22	40.3
7A	5	Failure (12.3)	12.7
7B	5	10	12.9
8A	6	Failure (12.5)	12.8
8B	5	13	13.5
9A	4	26	33.5
9B	4	22	34.2
10A	5	22	35.1
10B	5	16	33.6

Table 4.6: First Crack Events

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Figure 4.3: Test region crack map (top view) for Specimen 5A through Specimen 6B

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

(b) Top View with Strain Gage Location Relative to Cracks

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(a) Side View

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(a) Side View

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5 ANALYSIS OF DATA

5.1 Introduction

This chapter presents the analysis of data for all specimens presented in Chapter 4. The analysis includes the following items: (i) maximum tensile stress in reinforcement; (ii) test/calculated stress ratio vs. coating thickness, test is the stress at failure and the calculated values is determined in accordance with the AASHTO LRFD (2004) Specification and the ACI 318-05 Code; (iii) deflections at failure load; (iv) average crack spacing; (v) average crack widths; and (vi) failure conditions.

5.2 Maximum Tensile Stress in Reinforcement

The maximum tensile stress for the epoxy-coated reinforcement is presented in Table 5.1. There are two values of stress presented in this table, ***Inferred from Strain Failure Stress*** and ***Equilibrium Calculated Failure Stress***, which differ in the method by which they were calculated. The Stress vs. Strain curves Figure 3.2; data for Bar 5-1 and Bar 9-1 tension tests were used as approximate averages for all No. 5 and No. 9 bars respectively) were used for the appropriate bar size to convert Average Failure Strain (Table 4.2) to the ***Inferred from Strain Failure Stress***, using linear interpolation between data points. The ***Equilibrium Calculated Failure Stress*** was obtained by using beam theory to analyze the beam tested at the Failure Load given in Table 4.1 and determine the Failure Stress in the reinforcement for each beam. The following properties and assumptions were used to determine this ***Equilibrium Calculated Failure Stress***:

- Concrete: Hogenstad's stress vs. strain approximation for concrete was used to determine the concrete contribution in the compression zone (bottom of beam). The compressive strength (f'_c) for each beam was determined using the data from the 4 in. by 8 in. concrete cylinder compressive strength tests (discussed in Section 3.3.1 and shown in Figure 3.1) using the same age of the beam's test date (linearly interpolated between concrete cylinder ages if necessary). Also, a 0.95 reduction factor for concrete strength was used to account for the use of 4 in. by 8 in. cylinders, which tend to show higher strengths than a 6 in. by 12 in. concrete cylinder compressive strength test.
- Steel: Three stress-strain relationships were used for the epoxy-coated steel reinforcement properties. First, the modulus of elasticity for steel ($E_s = 29,000$ ksi) was used for the linear-elastic range. For the yield plateau region, the strain gage Stress vs. Strain curves (Figure 3.2; data for Bar 5-1 and Bar 9-1 tension tests used as approximate averages for No. 5 and No. 9 bars respectively) were used for the appropriate bar size. There were two specimens (Specimen 5A and 5B; No. 5 bars tested) that failed while the steel reinforcement was in the strain-hardening region (or Past Yield). Since there is no strain gage data for this region, the extensometer Stress vs. Strain curve (Figure 5.1, Bar 5-3 data was used due to the extensometer slipping for Bar 5-1, which was used for the yield plateau) was used. Near the bottom of the

beam, No. 6 longitudinal black steel reinforcement was present, and the modulus of elasticity for steel ($E_s = 29,000$ ksi) was used for the stress-strain relationship since it remained in the linear-elastic range prior to failure.

- **Beam Geometry:** The effective depth, d , of the epoxy-coated reinforcement location and the concrete compression zone width, b_{bottom} , is listed in the as-built data listed for each specimen in Table 3.10 and discussed in Section 3.5. The effective depth for the longitudinal black steel reinforcement location, as discussed in Section 3.5, is 2.75 in. from the bottom of the beam for Specimens 1A through 2B and 2.375 in. from the bottom of the beam for Specimens 3A through 10B. For visualization purposes the typical splice-region cross-sections for Single Splice and Bundled Bar beams is shown in Figure 3.4, where two spliced bars is the equivalent of one continuous bar as far as strength of the beam is concerned.

It is observed in Table 5.1 that, for each bar size that was designed for $1/2 f_y$, the ***Equilibrium Calculated Failure Stresses*** are similar when comparing the Bundle Bar specimens (Specimens 1A through 2B and Specimens 3A through 4B) with the Single Splice Specimens (Specimens 7A through 8B and Specimens 9A through 10B, respectively).

The Miller et al. Failure Stress presented in Table 5.1 refers to the failure stress of No. 6 epoxy-coated bars in beam-end specimens, fabricated and tested in accordance with ASTM A 944 (Miller et al (2003)). The first letter in the Specimen Number notation, either B or C, refers to the deformation pattern of the bar, which notation will be used to identify them from now on in this report. It should be noted that the relative rib areas for the Miller et al. No. 6 bars of deformation patterns B and C are 0.093 and 0.070 respectively. This is comparable to the relative rib areas of the No. 5 and No. 9 bars used in this project, which are 0.064 and 0.085 respectively (presented in Table 3.5).

5.3 Test/Calculated Stress Ratio vs. Coating Thickness

This section discusses a comparison of Test/Calculated Stress Ratio vs. Coating Thickness for all the specimens in Table 5.1. Test values are inferred from load measurements as explained in Section 5.2. Only the ***Equilibrium Calculated Failure Stress*** from Section 5.2 is used as the Test Stress shown in Figure 5.2 and Figure 5.3. Calculated values of stress are determined from two Code Specifications, AASHTO LRFD (2004) and the ACI 318-05 Building Code for Structural Concrete using the as-built properties listed in Table 3.10 and discussed in Section 3.5. Both bond specifications for development length and splices of epoxy-coated bars were described in detail in Section 2.3. It should be noted that the Class C or Class B factors were not used in determining the Calculated values of stress for either the AASHTO LRFD (2004) (AASHTO 5.11.5.3.1) Specification or the ACI 318-05 (ACI 318-05 12.15.1) Building Code, respectively. This is because these factors are not for strength considerations, but speak to the brittle mode of failure when all the splices are at the same location. The Coating Thickness measurements reported in Chapter 3 are used to determine the coating

thicknesses shown in Figures 5.2 and 5.3. The horizontal Coating Thickness measurements used in Figure 5.2 and Figure 5.3 are the average of all the Splice-Region Epoxy-Coating Thickness Measurements for each specimen presented in Table 3.7.

The specifications were used to establish a relationship between stress and bond length. The expressions for the AASHTO 2004 Calculated Stress ($f_{sAASHTO}$) are given in Equations (5.1) through (5.3) below. For Bundled Bar specimens, a Bundled Bar factor of **1.20** was used as one of the modification factors (Mod. Fact.) in the Calculated Stress ($f_{sAASHTO}$) equation, as described in Section 5.11.2.3 of AASHTO LRFD (2004).

$$f_s = \frac{\ell_a \sqrt{f'_c}}{1.25 A_b (\text{Mod. Fact.})} \quad (5.1)$$

$$f_{s2} = \frac{\ell_a}{0.4 d_b (\text{Mod. Fact.})} \quad (5.2)$$

$$f_{sAASHTO} = \text{Min.}(f_s, f_{s2}) \quad (5.3)$$

The results of the comparison between test/calculated stress ratio using the AASHTO LRFD (2004) vs. coating thickness are shown in Figure 5.2. On the vertical axis a ratio of 1.0 indicates that the test value and the calculated value were the same. Thus, Figure 5.2 indicates that the test value was greater than the calculated value from the specification for all the specimens in Table 5.1. It should be noted that the four specimens designed to achieve a yield stress at failure (shown filled) with coating thickness of 20 mils or more all achieved the yield stress. Furthermore, the eight single splice specimens designed to achieve $1/2 f_y$ indicated that the ones with thicker coatings (designations No. 5, $1/2 f_y$ and No. 9, $1/2 f_y$ in Figure 5.2) achieved at least equal or higher stress at failure than those with the thinner coating. The specimens with bundled bars showed a similar trend (designations No. 5, B.B., $1/2 f_y$ and No. 9, B.B., $1/2 f_y$). The specimens tested by Miller et al. also had ratios of test to calculated stress at failure greater 1.0. From this comparison, it can be concluded that increasing the coating thickness up to 21 mils did not result in ratios of test to calculated values at failure less than 1.0.

The expression from the ACI 318-05 Code was also rearranged to determine calculated stresses as shown in Equation (3.1). This equation is given in Section 3.2.2 of this report and is not repeated here. For Bundled Bar specimens, the ACI 318-05 bundled bar factor is used as described in Section 3.2.2. The results of the comparison between test/calculated stress ratio using the ACI Code vs. coating thickness are shown in Figure 5.3. The comparison showed that the ACI calculated values resulted in ratios higher than 1.0. Individual values were slightly less than those calculated using the AASHTO Code, except for the No. 9 bundled bars that were slightly lower for ACI than for AASHTO. The largest decrease was observed in the case of No. 5 single splice specimens designed for f_y . Table 5.2 summarizes the comparison of Test/Calculated ratio for the AASHTO LRFD (2004) and the ACI 318-05 Code. The data are presented by Design Group with the range Percent Increase in Stress Ratio reflecting the minimum and maximum for each group. It should be noted that the largest increase for the AASHTO vs. ACI comparison

(24% for No. 5, f_y) was less than the smallest increase for the ACI vs. AASHTO comparison (27% for the No. 9 bundle, $\frac{1}{2} f_y$).

5.4 Deflections at Failure Load

The deflection measurements are presented in detail in Section 4.4. In this section the measured deflections at failure are graphed in Figure 5.4 through Figure 5.9. In these figures, the vertical axis represents the LVDT deflection measurements corresponding to the various locations along the length of the beam plotted on the horizontal axis. Downward deflections are graphed as positive measurements while negative deflections are upward deflection. The origin of the horizontal axis is located at the centerline of the beam. Each figure presents the displacements for beams in each design group. The deflection measurements in these figures are the table values in Chapter 4 corrected to account for support settlement when this support settlement was monitored. For the specimens plotted in Figure 5.4, support settlement was not monitored and thus the deflection plots are not corrected.

In the figures the closed symbols denote specimens with bars that have a coating thickness range of 12-15 mils. The open symbols denote specimens with coated bars that have a coating thickness range of 18-21 mils. As can be seen from the figures, except for Specimens 5A and 5B, the specimens containing bars with a coatings thickness range of 18-21 mils did not show significantly different deflections at failure when compared to those for the companion specimens with bars that have a coating thickness range of 12-15 mils. The difference in the displacements between Specimen 5A and 5B was explained in Section 4.4 and was not an effect associated with the thickness of coating since both beams had bars with similar coating thickness.

5.5 Average Crack Spacing

Table 5.3 contains the average flexural crack spacing for all the specimens tested. The data are presented by Design Group for each range of coating thickness, 12-15 mils and 18-21 mils. The last column in this table gives the change in crack spacing determined as the difference of the average crack spacing between the 18-21 mils specimens and the 12-15 mils specimens divided by the average crack spacing in the 12-15 mils specimens. Thus positive numbers in this column represent increases in the average crack spacing for the 18-21 mils specimens. The actual measurements of crack spacing for each specimen can be found in Table 4.4. The specimens with splices designed for f_y were constructed only with bars that have a coating thickness range of 18-21 mils, as the intent was to evaluate the specification for the proposed coating thickness increase.

From Table 5.3, a review of the data indicates that for the No. 5 Bundled Bar specimens designed for $\frac{1}{2} f_y$, the increase in coating thickness resulted in a 4% **reduction** in the average crack spacing, for thicker coating thicknesses. For the No. 9 Bundled Bar specimens designed for $\frac{1}{2} f_y$, the increase in coating thickness resulted in a 14% **reduction** in the average crack spacing for thicker coating thicknesses.

For the No. 5 Single Splice specimens designed for $1/2 f_y$, the increase in coating thickness corresponded to a 23% **increase** in the average crack spacing for thicker coating thicknesses. For the No. 9 Single Splice specimens designed for $1/2 f_y$, the increase in coating thickness resulted in **no change** of average crack spacing, 0%, for thicker coating thicknesses. It appears as if the increase in coating thickness affected the bond strength of the smaller diameter bars more than the larger diameter bars. The reduction in bond strength can be associated with longer length of bar required to develop the same stress and thus produce cracking.

Table 5.4 contains the average flexural crack spacing for specimens designed for $1/2 f_y$ comparing Single Splice (Non-Bundled) to Bundled specimens. This comparison can be made due to similar failure stresses for each bar size that was designed for $1/2 f_y$, as observed in Table 5.1. The data are presented by Comparison Group for each bar configuration, Single Splice and Bundled. The last column in this table gives the change in crack spacing determined as the difference of the average crack spacing between the Bundled specimens and the Single Splice specimens divided by the average crack spacing in the Single Splice specimens. Thus positive numbers in this column represent increases in the average crack spacing for the Bundled specimens.

A review of the data in Table 5.4 indicates that the bundling of the bars for No. 5 diameter bars with 12-15 mils of epoxy-coating thickness resulted in a 7% **reduction** in the average crack spacing, for Bundled Bar specimens. The bundling of No. 9 bars with 12-15 mils of epoxy-coating thickness resulted in a 9% **increase** in the average crack spacing for Bundled Bar specimens. The bundling of No. 5 bars with 18-21 mils of epoxy-coating thickness is associated with a 28% **reduction** in the average crack spacing for Bundled Bar specimens. The bundling of the bars of No. 9 bars with 18-21 mils of epoxy-coating thickness resulted in a 6% **reduction** in the average crack spacing for Bundled Bar specimens. Bundling of bars resulted in an increase of the average crack spacing compared to that in Single Splice specimens only in the case of the No. 9 bar specimens with 12-15 mils coating thickness. In the other three cases considered, No. 5 bar specimens with 12-15 mils coating thickness, No. 5 bar specimens with 18-21 mils coating thickness and No. 9 bar specimens with 18-21 mils coating thickness, bundling of bars resulted in a decrease in the average crack spacing when compared to similar Single Splice specimens. The largest reduction was observed in the No. 5 bar specimens with coating thickness in the range of 18-21 mils.

5.6 Average Crack Widths

The average flexural crack widths selected for analysis are presented in Table 5.5. The data are presented by Design Group for each range of coating thickness, 12-15 mils and 18-21 mils, for selected Applied Load levels at which the measurements were taken. The selection of the Applied Load levels for each Design Group in the table was discussed in Section 4.7. The Average Design Group Flexural Crack Width listed in Table 5. under the corresponding thickness of coating was determined as follows: first, the average of the **A** and **B** crack widths listed in Table 4.5 for each specimen was calculated; next, for specimens within the same Design Group with similar thickness of coating an average of

the flexural crack width calculated in the first step was taken; if only one specimen of a given coating thickness existed at the selected Applied Load for a given Design Group, the value for that specimen was listed as the average for Design Group. The % change determined as the difference of the average crack width of the 18-21 mils specimens and the 12-15 mils specimens divided by the average crack width in the 12-15 mils specimens is given in Table 5.5 under the column heading Change in Crack Width. Thus positive numbers in this column represent increases in the average crack widths observed for the 18-21 mils specimens. The actual measurements of crack widths for each specimen can be found in Table 4.5.

As noted in Section 4.7, the crack widths given are for the highest Applied Load level listed in Table 4.5. The following are observations of the coating thickness effect observed in each design group:

- **No. 5 bundle, $1/2 f_y$:** In Table 5.5, the increase in epoxy-coating thickness made a significant difference resulting in an *increased* average crack width, 23%, for thicker coating thicknesses. It should be noted that Specimen 1B was not included in this comparison due to data not taken at the Applied Load available for comparison. However, it can be observed from Table 4.5 at the Applied Load level of 12 kips that its average crack width is slightly lower than its companion specimen, Specimen 1A.
- **No. 9 bundle, $1/2 f_y$:** In Table 5.5, the increase in epoxy-coating thickness made a significant difference resulting in a *reduction* in the average crack width, -29%, for thicker coating thicknesses. This observation for the average crack width is consistent with observations regarding flexural crack spacing for this design group (discussed in Section 5.5). It should be noted that Specimen 3A was not included due to data not taken at the compared Applied Load. However, it can be observed from Table 4.5 at the Applied Load level of 28 kips that its average crack width is comparable to its companion specimen, Specimen 3B. It should also be noted that for Specimen 4B, data at the Applied Load level was not used in the comparison because the Applied Load level was different, 40 kips. Despite difference in Applied Load level corresponding to the measurement, the average crack widths are similar as observed in Table 4.5 for Specimen 4A at 36 kips and Specimen 4B at 40 kips.
- **No. 5, f_y :** In Table 5.5, only the average crack width for Specimen 5A is presented. Specimen 5B has a different Applied Load level, 13 kips; however, the average crack widths **A** in Table 4.5 are similar for Specimen 5A and 5B at Applied Loads of 14 kips and 13 kips respectively. Also, the average crack widths at the common Applied Load level of 7 kips are comparable for these two specimens (Table 4.5).
- **No. 9, f_y :** In Table 5.5, the average crack width for Specimen 6A and 6B is presented. It should be noted that average crack width **A** for Specimen 6A in

Table 4.5 is slightly lower than the other average crack widths for these two specimens.

- **No. 5, $1/2 f_y$:** In Table 5.5, the increase in epoxy-coating thickness made a significant difference resulting in an *increased* average crack width, 23%, for thicker coating thicknesses. This observation for the average crack width is consistent with observations regarding flexural crack spacing for this design group (discussed in Section 5.5). It should be noted that Specimen 7A was not included in this comparison due to data not taken at the Applied Load available for comparison. However, it can be observed from Table 4.5 for Specimen 7A and 7B at the Applied Load of 8.5 kips and 8 kips respectively that the average crack width of Specimen 7A is slightly higher than that of its companion Specimen 7B. It should also be noted that a comparison was not conducted between Specimen 7A and 8A because the Applied Load level at which measurements were available was different for the two specimens as shown in Table 4.5.
- **No. 9, $1/2 f_y$:** In Table 5.5, the increase in epoxy-coating thickness resulted in a *reduction* in the average crack width, -13%, for thicker coating thicknesses. The highest Applied Load level for these specimens listed in Table 4.5 was not used because there was not a common Applied Load for comparison between the 12-15 mils and 18-21 mils specimens. It should be noted that Specimen 9A was not included due to data not taken at the compared Applied Load level. However, it can be observed from Table 4.5 for Specimens 9A and 9B at the Applied Load level of 22 kips and 20 kips respectively that the average crack widths are similar. It should also be noted that for Specimen 10B, data at the Applied Load level was not used in the comparison because the average **B** crack widths for this specimen are half than its average **A** crack widths. However the average **A** crack widths for Specimen 10B are similar to the average crack widths for Specimen 10A.

In summary, the trend for No. 5 bars (both Bundled and Single Splice specimens) is that the increase in epoxy-coating thickness resulted in an *increased* average crack width for thicker coating thicknesses. The observed trend for No. 9 bars (both Bundled and Single Splice specimens) is that the increase in epoxy-coating thickness resulted in a *reduction* in the average crack width for thicker coating thicknesses. The observed reduction for No. 9 Bundled Bar specimens as the coating thickness was increased was more significant than for No. 9 Single Splice specimens. It should be noted that the Applied Load level is not necessarily the same Applied Load to Failure Load ratio; therefore direct comparisons cannot be made between different Design Groups. Only trends can be compared.

5.7 Failure Conditions

The failure conditions, in terms of stress in the reinforcement and cracking, for each specimen are summarized in Table 5.6. At failure, the Steel Reinforcement was in the

elastic range or the yield plateau for all specimens except 5A and 5B. These two specimens had full length splices designed using the ACI 318-05 Code. At failure, strain gages in these two specimens were no longer in service and are described as Past Yield in Table 5.6. A splice failure is the mode of failure for all the beams tested, but Mode of Failure refers to the type of splice failure that occurred in each beam. There are three different Modes of Failure observed: Top Splitting, Side Splitting, or Top Cover. Top Splitting refers to the top concrete cover failing due to splitting; Side Splitting refers to the side concrete cover failing due to splitting; and Top Cover refers to the top concrete cover popping off at the line of reinforcement. The notation Top/Side Splitting is when both Top Splitting and Side Splitting occurred at the same time.

5.8 Findings

The main findings from the experimental data analysis conducted in this chapter are as follows:

- (i) For each bar size specimen designed for $1/2 f_y$, the ***Equilibrium Calculated Failure Stresses*** were similar when comparing the Bundle Bar specimens with the Single Splice specimens;
- (ii) Increasing the coating thickness up to 21 mils did not result in ratios of test to calculated values at failure less than 1.0 for both AASHTO LRFD (2004) and ACI 318-05 specifications;
- (iii) The specimens containing bars with a coatings thickness range of 18-21 mils did not show an appropriate difference in deflections at failure when compared to those of the companion specimens with bars that having a coating thickness range of 12-15 mils;
- (iv) For the No. 9 Bundled Bar specimens designed for $1/2 f_y$, the increase in coating thickness resulted in a ***reduction*** in the average crack spacing and average crack width for thicker coating thicknesses;
- (v) For the No. 9 single bar splices the increase in coating thickness resulted in a reduction in the average crack width. The reduction was less significant than that observed in the No. 9 Bundled Bar specimens. Furthermore, the increase in coating thickness resulted in no change in the average crack spacing.
- (vi) For the No. 5 Single Splice specimens designed for $1/2 f_y$, the increase in coating thickness resulted in an ***increase*** of average crack spacing and average crack width for thicker coating thicknesses;
- (vii) The bundling of the bars for No. 5 bars with 18-21 mils of epoxy-coating thickness resulted in a ***reduction*** of average crack spacing for Bundled Bar specimens.

The impact of these findings on current code specifications are discussed in Chapter 6. Also in Chapter 6, a field instrumentation plan is proposed for possible implementation by the Indiana Department of Transportation (INDOT) in the construction of a bridge deck in Indiana.

Specimen No.	Inferred from Strain Failure Stress (ksi)	Equilibrium Calculated Failure Stress (ksi)
1A	62.2	62.4
1B	49.0	61.9
2A	61.9	62.7
2B	57.4	62.3
3A	49.2	56.9
3B	50.1	61.6
4A	51.8	58.5
4B	50.3	59.4
5A	62.6	78.3
5B	---	79.8
6A	69.4	73.8
6B	62.2	69.9
7A	46.3	61.9
7B	51.1	62.1
8A	51.9	62.4
8B	57.4	63.0
9A	46.6	59.0
9B	51.9	59.8
10A	52.9	62.2
10B	52.8	58.9
Miller et al. Failure Stress Below		
B12A	---	47.6
B12B	---	42.0
B12C	---	48.9
B15A	---	42.1
B15B	---	40.9
B15C	---	43.4
B18A	---	41.0
B18B	---	41.7
B18C	---	42.2
B7A	---	41.1
B7B	---	44.1
B7C	---	42.2
C12A	---	40.7
C12B	---	42.5
C12C	---	44.7
C15A	---	39.7
C15B	---	38.1
C15C	---	36.2
C18A	---	37.6
C18B	---	37.8
C18C	---	41.6
C7A	---	39.2
C7A-B	---	44.9
C7B	---	47.4
C7C	---	43.5

Table 5.1: Summary of Maximum Tensile Stress in Reinforcement

Design Group	Percent Increase in Stress Ratio for	
	ACI vs. AASHTO	AASHTO vs. ACI
No. 5 bundle*, $1/2 f_y$	---	6% - 19%
No. 9 bundle*, $1/2 f_y$	27% - 40%	---
No. 5, f_y	---	24%
No. 9, f_y	---	6% - 9%
No. 5, $1/2 f_y$	---	15% - 17%
No. 9, $1/2 f_y$	---	4% - 9%

*Bundle is a three-bar bundle

Table 5.2: Test/Calculated Stress Ratio Comparison for ACI 318-05 vs. AASHTO 2004 Code Equations

Design Group	Average Design Group Flexural Crack Spacing (in.)		Change in Crack Spacing (%)
	12 mils	18 mils	
No. 5 bundle*, $1/2 f_y$	7.22	6.91	-4
No. 9 bundle*, $1/2 f_y$	8.15	7.02	-14
No. 5, f_y	---	7.94	---
No. 9, f_y	---	6.37	---
No. 5, $1/2 f_y$	7.77	9.56	23
No. 9, $1/2 f_y$	7.48	7.47	0

*Bundle is a three-bar bundle

Table 5.3: Coating Thickness Effect on Flexural Crack Spacing

Comparison Group	Average Design Group Flexural Crack Spacing (in.)		Change in Crack Spacing (%)
	Single Splice	Bundled*	
No. 5, 12 mils	7.77	7.22	-7
No. 9, 12 mils	7.48	8.15	9
No. 5, 18 mils	9.56	6.91	-28
No. 9, 18 mils	7.47	7.02	-6

*Bundled has three-bar bundles

Table 5.4: Bundling Effect on Flexural Crack Spacing for $1/2 f_y$ Specimens

Design Group	Applied Load (kips)	Average Design Group Flexural Crack Width (in.)		Change in Crack Width (%)
		12 mils	18 mils	
No. 5 bundle*, $1/2 f_y$	17.5	0.016	0.020	23
No. 9 bundle*, $1/2 f_y$	36	0.023	0.017	-27
No. 5, f_y	14	---	0.039	---
No. 9, f_y	26	---	0.017	---
No. 5, $1/2 f_y$	12	0.022	0.027	23
No. 9, $1/2 f_y$	20	0.015	0.013	-13

*Bundle is a three-bar bundle

Table 5.5: Coating Thickness Effects on Flexural Crack Widths

Specimen No.	Steel Reinforcement	Mode of Failure
1A	Yield Plateau	Top/Side Splitting
1B	Elastic	Top/Side Splitting
2A	Yield Plateau	Top/Side Splitting
2B	Elastic	Top/Side Splitting
3A	Elastic	Side Splitting
3B	Elastic	Side Splitting
4A	Elastic	Side Splitting
4B	Elastic	Side Splitting
5A	Past Yield	Side Splitting
5B	Past Yield	Top/Side Splitting
6A	Yield Plateau	Top Cover
6B	Elastic	Top Cover
7A	Elastic	Side Splitting
7B	Elastic	Top/Side Splitting
8A	Elastic	Side Splitting
8B	Elastic	Side Splitting
9A	Elastic	Top Cover
9B	Elastic	Top Cover
10A	Elastic	Top Cover
10B	Elastic	Top Cover

Table 5.6: Summary of Failure Conditions

Figure 5.1: Stress vs. Strain (Extensometer) curve for No. 5 steel reinforcing bars

Figure 5.2: Coating Thickness vs. Test/AASHTO Calculated Stress Ratio

Figure 5.3: Coating Thickness vs. Test/ACI Calculated Stress Ratio

Figure 5.4: Beam Deflection for Specimen 1A through Specimen 2B (these specimens are not corrected for support settlement)

Figure 5.5: Beam Deflection for Specimen 3A through Specimen 4B

Figure 5.6: Beam Deflection for Specimen 5A and Specimen 5B

Figure 5.7: Beam Deflection for Specimen 6A and Specimen 6B

Figure 5.8: Beam Deflection for Specimen 7A through Specimen 8B

Figure 5.9: Beam Deflection for Specimen 9A through Specimen 10B

6 SUMMARY, FINDINGS AND IMPLEMENTATION

6.1 Summary

In Chapter 1, the introduction of the problem and the motivation of the study were presented. The main objective of the research was to evaluate the bond strength of splices of epoxy-coated bars with thickness of coating up to 18 mils and to provide design guidance on development length and splice length of epoxy-coated bars with thickness of coating in the same range.

In Chapter 2, the results of a literature review on the bond strength of epoxy-coated bars in tension are given. A few studies are noted as the motivation for the research conducted in this project. The background on bond between the concrete and the steel reinforcement was discussed. Also, the development length specifications from AASHTO LRFD (2004) and ACI 318-05 were reviewed.

Chapter 3 contains the experimental program conducted to evaluate the bond strength of reinforcing bars with extra coating thickness. The test program, material properties, instrumentation and construction, test setup and protocols, and data collection were presented. For the epoxy-coating thickness readings, a few trends were discussed for both the survey of all bars supplied and the splice-region measurements. The survey of all bars supplied showed that the average thickness per bar, for the majority of the bars, was within the specified range; however, some bars seemed to have different coating thicknesses on its opposing side and these bars had higher standard deviations than the bars that did not share this trend.

For each group of bars varying in epoxy-coating thickness (12-15 mils or 18-21 mils) and size (No. 5 or No. 9), it was noted that that the lowest average standard deviations are for the 12-15 mils No. 5 bars, and the highest average standard deviations are for the 18-21 mils No. 9 bars. For the splice-region measurements, it was noted that the mean coating thickness readings on the Ribs were always greater than in the Valleys. The intent to have at least 18 mils of epoxy-coating thickness was in general successful by specifying the coating thickness of a range of 18 to 21 mils.

In Chapter 4, the measured test data were presented, which include the (i) failure load, (ii) strain gage data, (iii) deflection data, (iv) crack patterns, (v) crack widths, and (vi) description of load events. In Chapter 5, the analysis of the experimental data presented in Chapter 4 and also relevant data found in the literature was conducted.

6.2 Findings

The main findings of the experimental program results reported in Chapter 4 and the analysis of those results conducted in Chapter 5 are:

- (i) For each bar size specimen designed for $1/2 f_y$, the ***Equilibrium Calculated Failure Stresses***, defined as the stress at failure obtained using

beam theory to analyze the beam tested at the Failure Load, were similar when comparing the Bundle Bar specimens with the Single Splice specimens.

- (ii) Increasing the coating thickness up to 21 mils resulted in ratios of test to calculated values at failure greater than 1.0 when the calculated values were determined using the AASHTO LRFD (2004) Specifications and the ACI 318-05 Code. ACI 318-05 calculated values resulted in an average test/calculated ratio slightly lower than the average for test/calculated ratios using the AASHTO LRFD (2004) Specifications. The No. 9 bundled bars however, were greater for ACI than for AASHTO. The test results support the use of the AASHTO LRFD Specifications to determine splice and development length of epoxy-coated bars with thickness of coating up to 18 mils on the basis of test/calculated ratios. However, the use of the ACI 318-05 development length specifications is recommended instead of the AASHTO LRFD (2004) Specifications. This recommendation is based on the fact that the ACI 318-05 Code in the bond provisions incorporates the effects of cover or transverse reinforcement whereas the AASHTO LRFD bond provisions does not. These two parameters are critical in the case of splitting failures associated with high bearing stresses as results from the deformations bearing against the concrete. Bearing of the deformations is the more significant contributor to the bond strength of epoxy-coated bars.
- (iii) The specimens containing bars with coatings thickness in the range of 18-21 mils did not show significantly different deflections at failure when compared to those of the companion specimens with bars having a coating thickness range of 12-15 mils.
- (iv) For the No. 9 Bundled Bar specimens designed for $1/2 f_y$, the increase in coating thickness resulted in a **reduction** in the average crack spacing and average crack width with thicker coatings.
- (v) For the No. 9 single bar splices the increase in coating thickness resulted in a reduction in the average crack width. The reduction was less significant than that observed in the No. 9 Bundled Bar specimens. Furthermore, the increase in coating thickness resulted in no change in the average crack spacing.
- (vi) For the No. 5 Single Splice specimens designed for $1/2 f_y$, the increase in coating thickness resulted in an **increase** of average crack spacing and average crack width for thicker coating thicknesses. The increase in coating thickness appeared to affect the bond strength of the smaller diameter bars more than the larger diameter bars. The increase in the crack spacing can be associated with a longer length of bar required to develop the same stress and thus produce cracking.

- (vii) Bundling of bars resulted in an increase of the average crack spacing compared to that observed in Single Splice specimens only in the case of the No. 9 bar specimens with 12-15 mils coating thickness. In the other three cases considered, No. 5 bar specimens with 12-15 mils coating thickness, No. 5 bar specimens with 18-21 mils coating thickness and No. 9 bar specimens with 18-21 mils coating thickness, bundling of bars resulted in a decrease in the average crack spacing when compared to similar Single Splice specimens. The largest reduction was observed in the No. 5 bar specimens with coating thickness in the range of 18-21 mils.

6.3 Implementation

6.3.1 Design Specifications

The use of the current provisions for development and splice length of epoxy-coated bars in tension in both the AASHTO LRFD (2004) and ACI 318-05 is supported by the test findings of the experimental program conducted in this study up to a coating thickness not to exceed 21 mils.

However, since the ACI 318-05 specifications consider the critical parameters of cover and transverse reinforcement, the authors encourage the Indiana Department of Transportation to use these provisions in the design of development and splice length of bars with coating thickness up to 18 mils. It is further recommended that these provisions be used in the design of an experimental concrete bridge deck using epoxy-coated bars with 18 mils of coating thickness.

INDOT 700 Committee is implementing the results of this study through a change in the specification for epoxy-coated bar thickness.

6.4 Monitoring Plan for Future Experimental Concrete Bridge Deck

A monitoring plan for a future concrete bridge deck to be built in Indiana using coated bars with thickness of coating up to 18 mils is presented in this section. The proposed monitoring plan includes three stages. The first stage will be prior to construction and includes sampling of the reinforcement, specifically coating thickness, and number of defects upon arrival to the job site and after placement. In addition, contractor, coater and epoxy powder manufacturer will be collected. Concrete specifications (w/c, air content and strength) will be documented together with epoxy-coated reinforcing steel specifications, quality assurance test results and inspection records including coating patching requirements and material and site storage length and conditions.

During construction in the second stage, the number of defects after casting of the concrete will be evaluated using the same technique as in the *“Methods of Corrosion Protection and Durability of Concrete Bridge Decks Reinforced with Epoxy-Coated Bars*

– *Phase I'* JTRP study. It is also proposed to determine relevant concrete material properties using established sampling techniques.

The third stage will involve the monitoring of the deck performance in-service conditions. It is proposed to evaluate the long term performance with a combination of regularly scheduled visits to obtain chloride levels in the concrete at various depths, survey for delaminating and obtain cores to inspect the concrete and the reinforcement. An excellent summary of available techniques is described in the Report NCHRP 10-37B, *“Protocol for In-Service Evaluation of Bridges with Epoxy-Coated Reinforcing Steel”* (Weyers 1995). Remote surveillance using corrosion (Embedded Corrosion Instrument, ECI) transducers in combination with a fully automated data acquisition system is also suggested for implementation as means to determine the conditions in the concrete in direct contact with the probe. The ECI has the capability to monitor linear polarization resistance, open circuit potential, resistivity, chloride ion concentration and temperature. These instruments have the capability to directly transmitting the data over a network connection.

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