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Performance-Related Specifications for Concrete Bridge Superstructures, Volume 4: Bond of Epoxy-Coated Bars with Thicker Coatings

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Final Report

PERFORMANCE-RELATED SPECIFICATIONS OF CONCRETE BRIDGE SUPERSTRUCTURES

FHWA/IN/JTRP-2001/8

Volume 4
Bond of Epoxy-Coated Bars with Thicker Coatings

By

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Conducted in Cooperation with the
Indiana Department of Transportation
and the
Federal Highway Administration

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Purdue University
West Lafayette, IN 47907
October 2002

INDOT Research

TECHNICAL *Summary*

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Performance Related Specifications (PRS) for Concrete Bridge Superstructures- A Four Volume Report

Introduction

The development of Performance Related Specifications (PRS) requires the identification of key performance levels for a given structural system. The first attempt to develop a methodology for PRS can be traced to 1980 when the Federal Highway administration (FHWA) instituted a new research program category. The main two objectives of the program were:

1) To provide a more rational basis for payment reduction plans.
2) To develop additional specifications related to the performance of flexible and rigid pavement structures.

In the early and mid-1980s, the FHWA, the National Cooperative Highway Research Program (NCHRP), and the American Association of State Highway and Transportation Officials (AASHTO) began a cooperative effort searching for supporting data needed for the development of PRS. The idea was to develop performance models that would allow relating the material and construction testing parameters collected at the time of construction to the future performance of the complete project. However, it was concluded that the existing databases were inadequate to derive the needed performance models. A known example of a PRS is the one developed for Portland Cement Concrete (PCC) pavements by Eres Consultants, Inc. and the FHWA (Darter et. al., 1998) in a cooperative effort. In this study, the overall objectives of a methodology for PRS were not completely fulfilled due to the lack of adequate supporting information in the existent databases to construct accurate performance predictive models. As a result, the proposed PRS was presented only as a methodology providing a more rational basis for payment plans.

The objective of the research study was to develop the essential components of a PRS for concrete bridge superstructures for application in the state of Indiana. The work conducted in this research project is presented in four volumes. Volume 1 summarizes the work conducted on the identification of performance levels and key parameters, and the development of acceptance criteria are addressed in Volume 1. The main objective of this volume is to present a proposed methodology for a PRS for concrete bridge superstructures. Volume 2 presents the research findings dealing with development of High-Performance Concrete (HPC) for applications in the bridge structures in the state of Indiana. The objective of the study presented in Volume 2 was to identify and develop concrete mixtures with adequate performance characteristics in terms of durability for the purpose of using these characteristics in performance-related specifications. Volume 3 summarizes the work conducted to investigate the behavior of fiber reinforced polymer (FRP) reinforced concrete structures with an emphasis on bond and shear. The main objective of this volume is to provide design guidelines for the use of FRP reinforcement in bridge superstructures. Volume 4 summarizes the results of an evaluation of the bond performance of epoxy-coated bars with a coating thickness up to 18 mils.

Findings

In this study emphasis has been placed on the development of a methodology for a Performance Related Specification, PRS, for concrete bridge superstructures. The implementation of the methodology, presented in the form of a user-friendly computer program in Volume 1 of this report, is project specific. It requires the mean and standard deviation (or definition of a probability distribution) of the input parameters for the performance predictive models. This is done for both the as-designed condition and the as-built condition of the structure. The contractor is expected to achieve certain level of compliance during the construction as dictated by the as-designed condition (which is defined based on the submitted design in compliance with agency specifications).

Based on performance predictive models, cost models, and statistical simulation, the methodology reports a relative as-built/as-designed Life-Cycle Cost (LCC). This relative LCC measures the level of compliance of the as-built structure with the design. The agency (INDOT) implementing the methodology could then consider the relative LCC in the form of a pay factor modifying the contractor's bid price.
Statistical simulation is conducted to evaluate the effects of the variations in the input parameters for the performance predictive models. The differences in the LCC for the as-designed and as-built elements come from the differences in the input parameters that are under the control of the contractor (referred to as quality characteristics). The framework of the proposed methodology has been fully developed and illustrated with four numerical examples in an initial case study of a simply supported reinforced bridge deck or slab.

The research effort described in Volume 2 of this report was divided in two phases. Phase I was focused on development of concrete mixtures optimized with respect to selected performance-related parameters. During this phase, ten optimum concrete mixes have been identified from 45 mixes in terms of compressive strength, Young's modulus of elasticity, rapid chloride penetration and chloride conductivity using a statistical design procedure. Through surface response methodology, 27 statistical models were developed for each of four parameters. Based on the models developed, 81 contour maps were generated, which indicated how performance of concrete varied in response to the change of dosages of binders at constant water-binder ratio. Based on the overlaid contour maps and the threshold values chosen for the properties of concrete, optimum concrete mixtures including Portland cement and the combinations with fly ash, silica fume and slag were identified.

In Phase II of the HPC study, the ten optimum mixtures were further evaluated with respect to mechanical properties and durability characteristics. Several different tests related to the evaluation of the resistance of concrete to chloride permeability were used: rapid chloride permeability test, chloride conductivity test, test for the resistance of concrete under DC electrical field, ponding test for the determination of the resistance of concrete to chloride penetration, and rapid test for the determination of diffusion coefficient from chloride migration. Tests related to the resistance of concrete to freezing & thawing, and scaling were also investigated. Other tests such as, the determination of drying shrinkage, and test for curing effects on the properties of high performance concrete were also evaluated in this research. Special emphasis was placed on determining and quantifying these parameters that control the ingress of the chloride ions.

Based on the results generated during this research, models have been developed that allow for prediction of certain mechanical and durability-related parameters related to the mixture composition. The parameters that can be predicted include strength, rapid chloride permeability (RCP) values, and chloride diffusion coefficient. Limited validation of these models was performed using field data provided by INDOT. The strength and chloride diffusion coefficient values generated by these models can serve as an input for the life-cycle costing (LCC) model described in Vol. 1 of this report

As summarized in Volume 3, experimental investigations were performed to specifically investigate the behavior of FRP reinforced concrete structures in both bond and shear. For the bond investigation, three series of beam splice tests were performed on specimens reinforced with steel, glass FRP, and aramid FRP to determine the effect of the different types of reinforcement on bond, cracking, and deflections. The test results indicate that the use of FRP reinforcement leads to lower bond strengths and, therefore, require longer development lengths. The specimen crack widths and deflections were substantially larger for FRP specimens than steel specimens due to

the significantly lower modulus of elasticity. Analysis of the test results resulted in recommendations for modifying the empirical development length equation of ACI 318-99 design code for use with FRP reinforcement.

For the shear investigation, two series of beam tests were conducted on specimens reinforced with steel, glass FRP, and aramid FRP to determine the effect of the different types of reinforcement on the concrete shear strength. All specimens did not contain transverse reinforcement. The test results show that the use of FRP reinforcement leads to lower concrete shear strengths than steel reinforcement for equal reinforcement cross-sectional areas (longitudinal reinforcement percentages). In addition, the test results point that the shear strength is a direct function of the longitudinal reinforcement stiffness. The test results further substantiated the findings that larger crack widths and deflections are achieved by FRP specimens relative to steel specimens due to the lower modulus of elasticity. Analysis of the test results resulted in recommendations for the calculation of concrete shear strength.

The experimental work on the bond performance of epoxy-coated bars with thickness up to 18 mils summarized in Volume 4 of the final report indicates that the current AASHTO requirements for development length of epoxy-coated bars could be extended to coating thickness of up to 18 mils.

Implementation

Based on the results from the research conducted on the framework for a PRS, it was concluded that the most practical implementation of the methodology had to consider the corrosion deterioration problem as the only distress determining/affecting the LCC of the structure. It was concluded that other distress indicators applied at "a section level" should be included in the framework of a PRS to give more integrity to the process of quality control. The needed software for the implementation of the proposed PRS has been provided to INDOT as part of this report. It must be noted that corrosion deterioration represents almost 50% of the problems of the current bridge infrastructure in Indiana.

As part of the implementation efforts for the part of the research dealing with HPC, a series of mathematical models were constructed that allow for the prediction of strength, rapid chloride permeability and chloride diffusion coefficient values based on the binder composition of the mixture.

The data generated using these models have been arranged in an Excel sheet, which allows the user to input desired minimum and maximum values of strength (at 28 days) and/or RCP values (at 56 days) and obtain binder combinations which yield/satisfy the desired input values. Binder system 1 refers to mixtures, which contain PC, SF and GGBS. Binder system 2 refers to mixtures, which contain PC, SF and FA. Binder system 3 refers to mixtures, which contain PC, GGBS and FA. The percentage increments of SF represented in the Excel worksheet are 0, 5 and 7.5 %. The percentage increments of FA and GGBS represented are 0, 20, 25 and 30 %.

The strength and chloride diffusion coefficient values determined for the 10 concrete mixtures tested in Phase II of the study were also used as input values for the LCC model described in Vol. 1 of this report. The LCC model was run for a single, simply supported span. The same type of data was also obtained from three existing Indiana bridges and the LCC model was re-run for these structures. The results indicate that LCC for all laboratory mixtures was lower than the LCC for standard INDOT class C concrete mixture. Furthermore, the LCC of the actual field mixtures was slightly higher than the LCC of standard class C mixture.

Currently, the ability of the models developed as a part of the HPC study to predict the actual properties of a field concrete is being validated on several QC/QA bridge jobs and a supplementary report summarizing the results of these evaluations is expected by June 2003.

Based on the research conducted on the use of FRP reinforcement, design and construction recommendations are provided that can be used in the design and construction of FRP reinforced bridge decks. These recommendations will be implemented in a JTRP study "Implementation of a Non-Metallic Reinforced Bridge Deck." This study will evaluate the design and construction recommendations in a prototype laboratory deck specimen as well as through a pilot field study that incorporates nonmetallic reinforcement in a bridge deck.

No change of the bond specifications is required to implement the use of up to #8 diameter deformed bars with epoxy-coating thickness up to 18 mils.

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16. Abstract

In Volume 4 of the final report, the results of an evaluation of the bond performance of epoxy-coated bars with a coating thickness up to 18 mils are presented. The experimental work on the bond performance of epoxy-coated bars with thickness up to 18 mils indicates that the current AASHTO requirements for development length of epoxy-coated bars up to #8 could be safely extended to coating thickness of up to 18 mils. Thus, no change to the bond specifications is recommended in order to implement the use of bars with diameters up to a #8 with epoxy-coating thickness up to 18 mils.

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1. INTRODUCTION

The thickness of the epoxy 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, September 2002). By increasing the upper limit of thickness to 18 mils, it has been found that the number of defects during construction decreases by approximately 50% (Samples, 2000). It has been suggested that increasing the epoxy thickness could decrease the bond strength between the reinforcing steel and the concrete (Samples, 2000). The focus of this task is to investigate the possibility of decreased bond performance due to thicker epoxy coatings. Deflections and cracking will be investigated since these are also related to the performance of the structure.

2. TEST PROGRAM

Three series of beams, A, B, and C, were tested. Series A was tested statically and Series B and C were tested under repeated loading. The detailed results of Series are available in Appelhans (2002). Table 2.1 summarizes the characteristics of each Series. The beams in Series A were purposely designed using a splice length well under AASHTO specifications to ensure that the beams would fail in bond. The goal in Series A is to establish differences in bond strength between bars with different coating thicknesses.

The loading schedule for the beams in Series B and C is intended to simulate traffic effects prior to one final cycle to failure. The beams in Series B and C are designed with splice lengths meeting AASHTO specifications. In Series B, the beams were under repeated loading to simulate traffic effects. Each beam was loaded to 1,000,000 cycles. At 100,000 cycle intervals, the testing was stopped to measure crack widths, count the number of cracks, and take photographs. After 300,000 cycles the beams were loaded until 1,000,000 cycles. Upon completion of the repeated loading port statically loaded to failure. The purpose of Series B and C is to find any differences in ultimate capacity, deflection behavior, and number of cracks between the 12 and 18 mil epoxy coated bars. The beam cross-sections and loading patterns were chosen to simulate a typical concrete bridge deck as found in Indiana.

Table 2.1: Specimen Characteristics

Series	Depth (in)	Bar Size	Concrete Strength (psi)	Splice Length (in)	Number of beams (uncoated)	Number of beams (12 mil)	Number of beams (18 mil)
A	8	#5	4000	12-14	2	2	2
B	8	#5	5000	30	----------	2	2
C	16	#8	5000	76	----------	2	2

3. MATERIAL PROPERTIES

Figure 3.1 shows the concrete development strength for Batches 1 and 2 in Series A. Figure 3.2 contains the 28-day development strength for Series B. Note that the actual 28-day strength (5160 psi) is very close to the design value (5000 psi). Figure 3.3 shows the strength gain for the mix used in Series C. Figure 3.4 contains the data for the steel reinforcement.

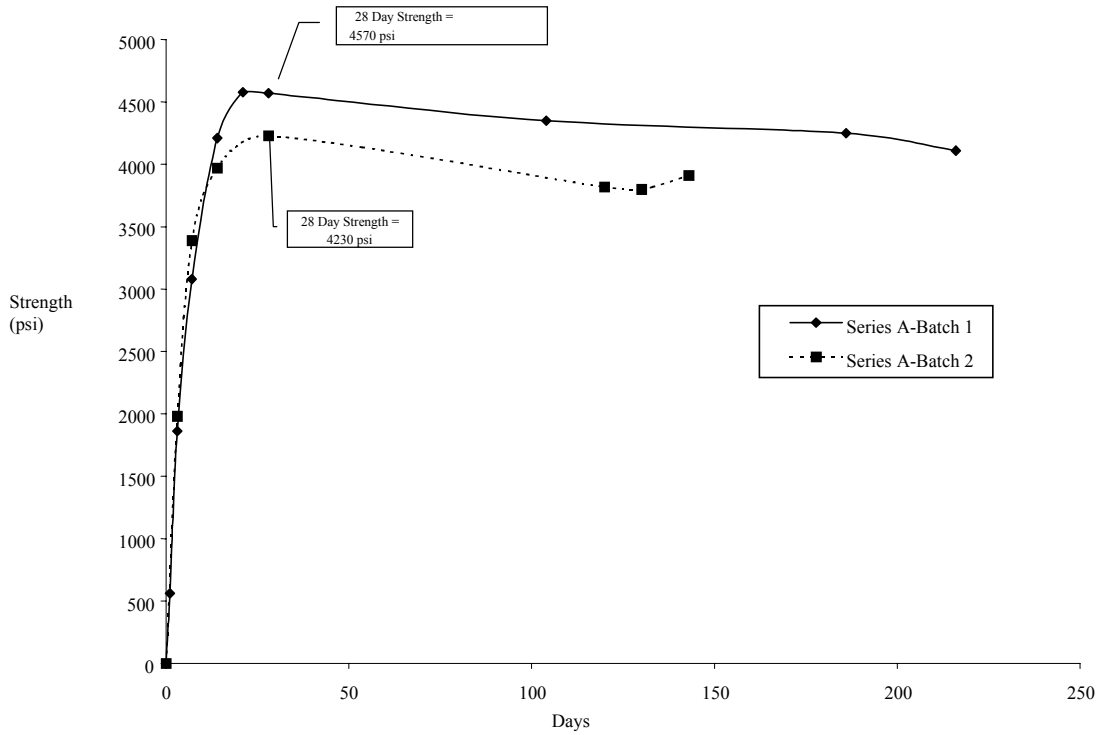

Figure 3.1 Concrete Strength Data for Batches 1 and 2 in Series A

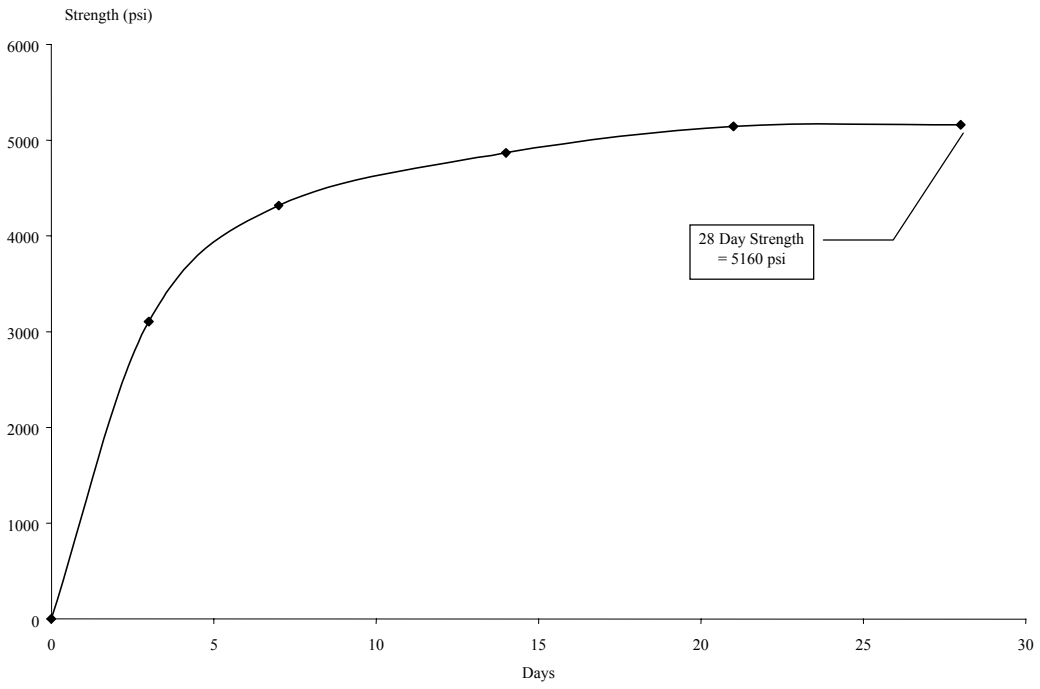

Figure 3.2 Concrete Strength Data for Series B

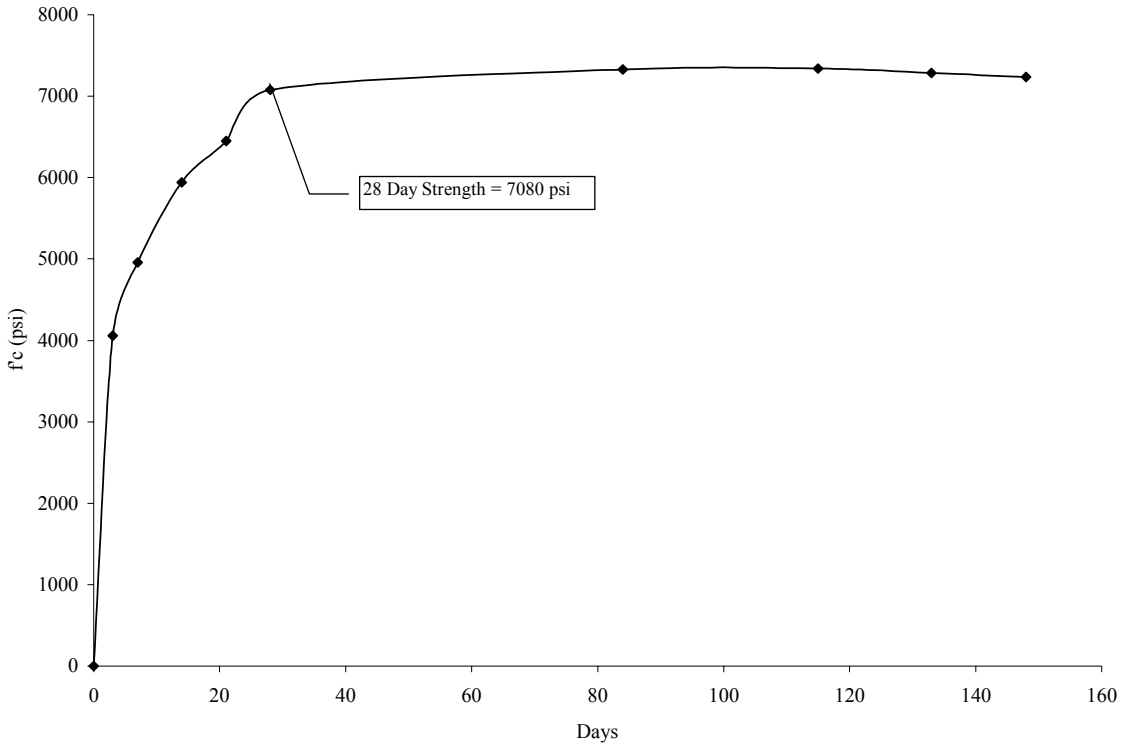

Figure 3.3 Concrete Strength Data for Series C

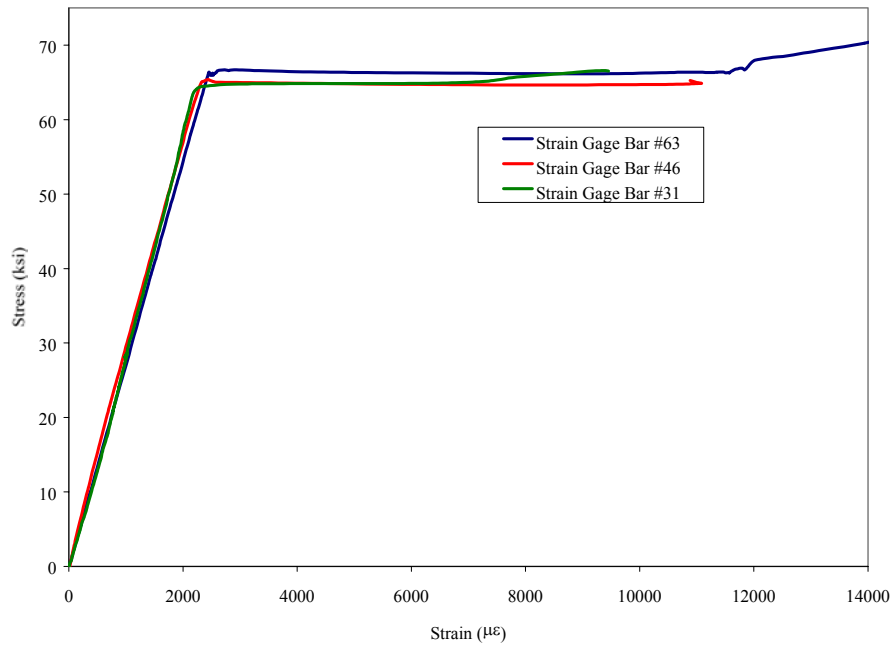

Figure 3.4 Steel Reinforcement: Series A, B, and C

4. SUMMARY OF WORK

The detailed results of Series A, B and C are shown in Table 4.1, and Figures 4.1, and 4.2 for Series A, Table 4.2, and Figures 4.3 and 4.4 for Series B, and Table 4.3 and Figures 4.5 and 4.6 for Series C. The tables contain a summary of the key experimental data.

Table 4.1 Summary of Test Results for Series A

Specimen Number	Epoxy Coating Thickness	Number of Cracks in the Splice Length	Splice Length (in.)	Failure Load (lbs)	Concrete Strength at Failure (psi)	Mode of Failure
A-U1	Uncoated	4	14	12600	4350	Bond
A-U2	Uncoated	3	12	12900	3820	Bond
A-E12-1	12 mil	4	14	12100	4250	Bond
A-E12-2	12 mil	3	12	12500	3800	Bond
A-E18-1	18 mil	4	14	12800	4110	Bond
A-E18-2	18 mil	3	12	11000	3911	Bond

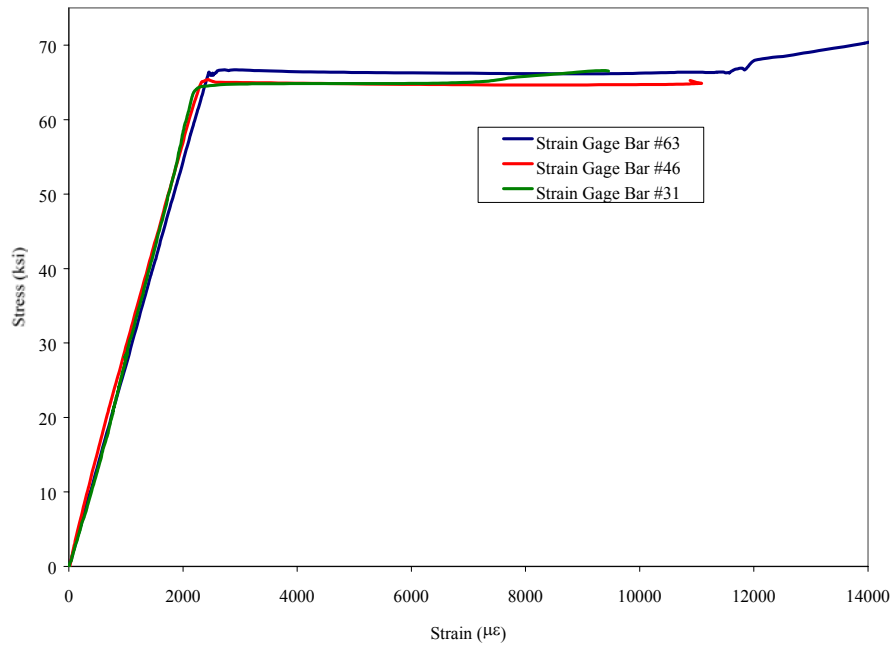

Figure 3.4 Steel Reinforcement: Series A, B, and C

4. SUMMARY OF WORK

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A-U2	Uncoated	3	12	12900	3820	Bond
A-E12-1	12 mil	4	14	12100	4250	Bond
A-E12-2	12 mil	3	12	12500	3800	Bond
A-E18-1	18 mil	4	14	12800	4110	Bond
A-E18-2	18 mil	3	12	11000	3911	Bond

Table 4.2 Summary of Test Results for Series B

Specimen Number	Epoxy Coating Thickness	Number of Cracks in the Splice Length	Splice Length (in.)	Failure Load (lbs)	Concrete Strength at Failure (psi)	Mode of Failure
B-E12-1	12 mil	4	30	12900	5340	Shear-Compression
B-E18-1	18 mil	4	30	12500	5190	Flexure
B-E12-2	12 mil	4	30	12200	5200	Flexure
B-E18-2	18 mil	4	30	12700	5120	Flexure

Table 4.3 Summary of Test Results for Series C

Specimen Number	Epoxy Coating Thickness	Number of Cracks in the Splice Length	Splice Length (in.)	Failure Load (lbs)	Concrete Strength at Failure (psi)	Mode of Failure
C-E12-1	12 mil	8	76	92500	7280	Shear
C-E18-1	18 mil	8	76	99300	7240	Shear
C-E12-2	12 mil	9	76	96600	7340	Shear
C-E18-2	18 mil	9	76	91000	7330	Shear

The figures show load against tip-deflection behavior for all the specimens in Series A and B, and load against midspan-deflection in Series C. A comparison of the results for the 12-mil coated and 18-mil coated reinforced beams shows little difference in performance. The beams with the uncoated reinforcement were both stronger and more ductile at failure in Series A. This is expected and accounted for by both ACI and AASHTO codes. The beams in Series B with code splice lengths showed a satisfactory performance. The results of Series C confirmed the findings of Series B for specimens reinforced with #8 bars.

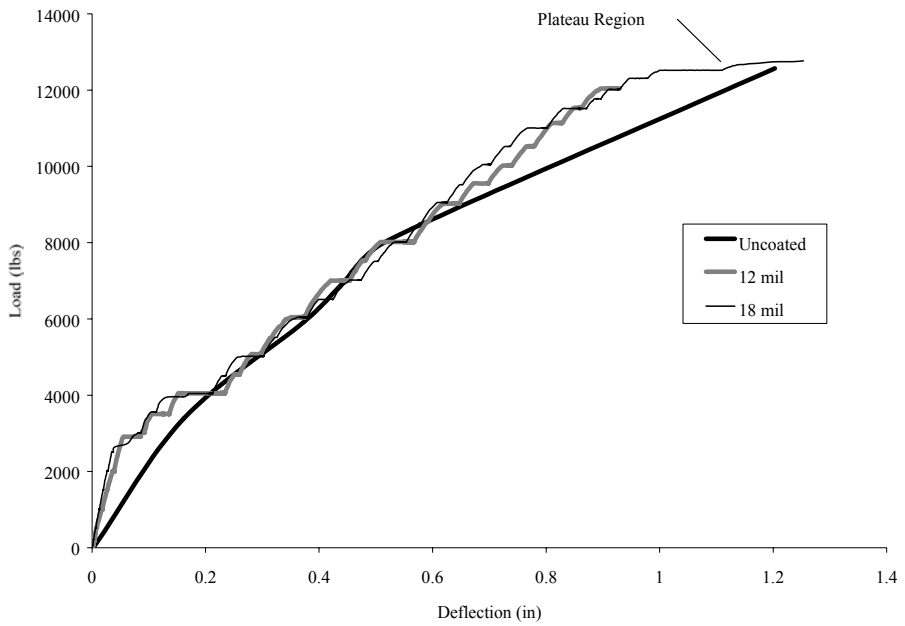

Figure 4.1 Load vs Tip-Deflection for Series A-Batch#1 Specimens

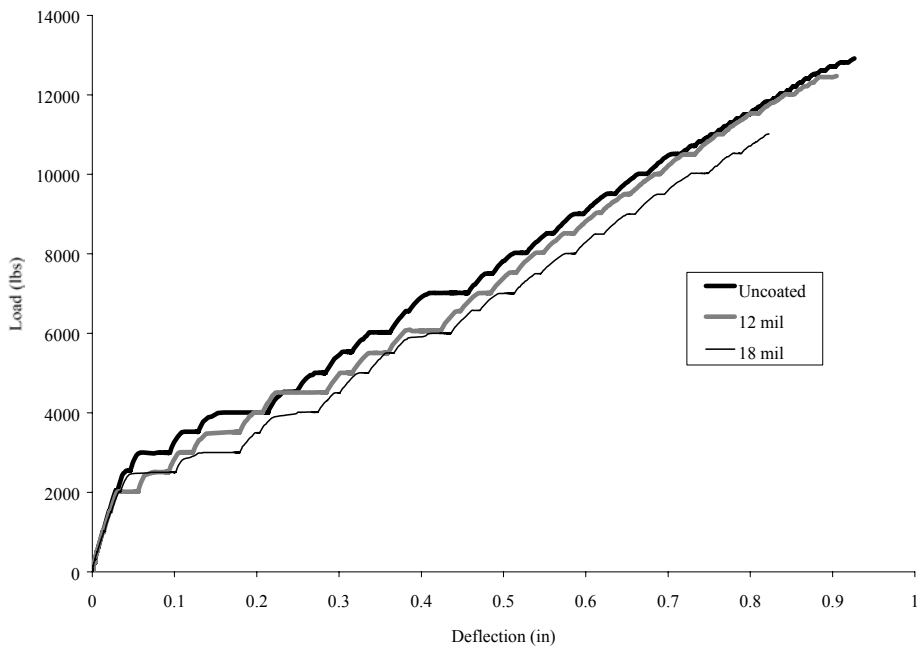

Figure 4.2 Load vs Tip-Deflection for Series A-Batch#2 Specimens

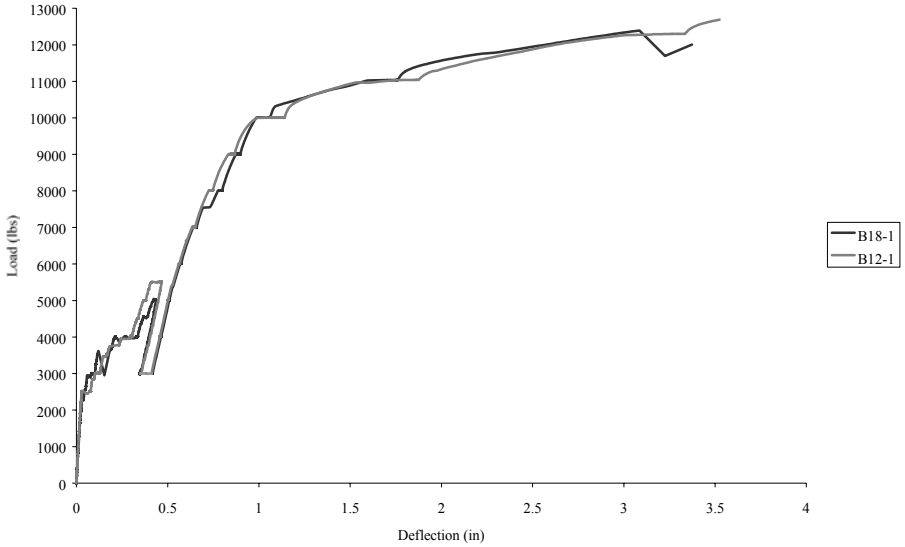

Figure 4.3 Load vs Tip-Deflection for Series B-Batch#1 Specimens

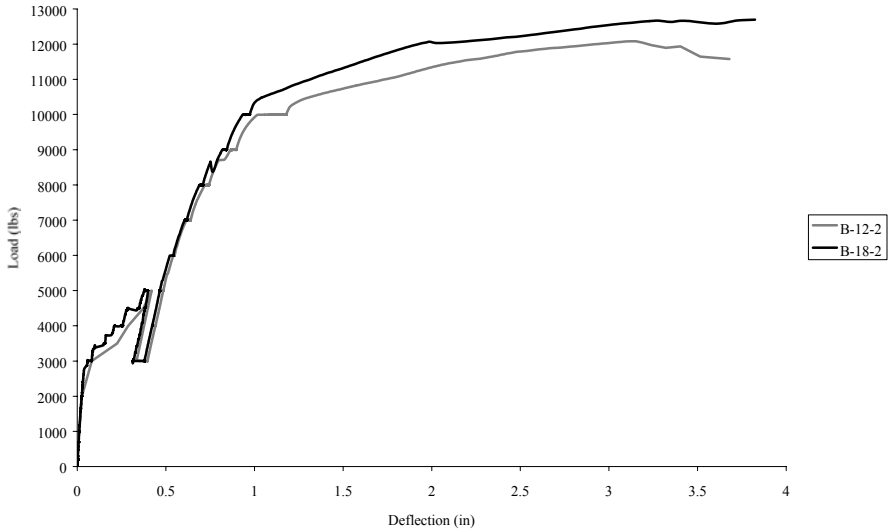

Figure 4.4 Load vs Tip-Deflection for Series B-Batch#2 Specimens

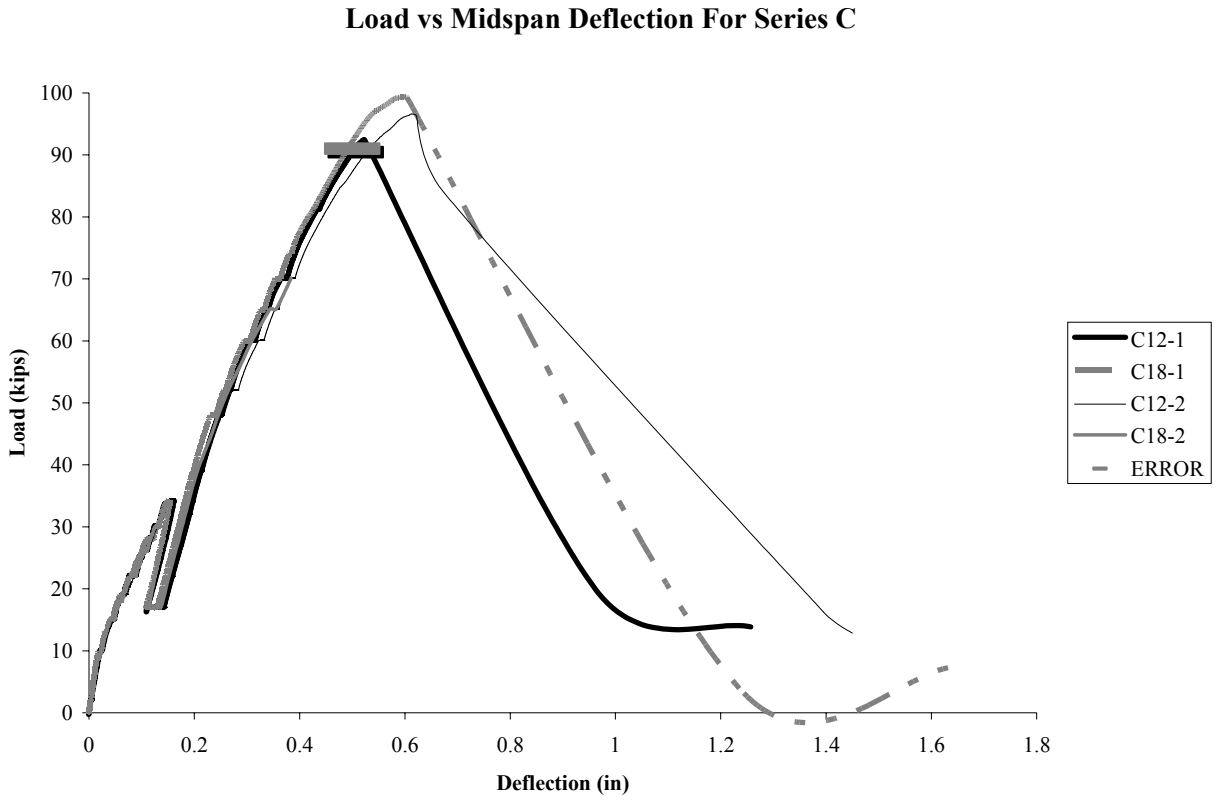

Figure 4.5. Load vs. mid-span deflection for Series C

5. FINDINGS

Based on the results from the experimental program conducted to date, it can be concluded that the current AASHTO requirements for development length of epoxy-coated bars can be extended to coating thickness of up to 18 mils.

6. REFERENCES

1. Samples, L. M., and Ramirez, J. A. (2000), *"Field Investigation of Concrete Bridge Decks in Indiana,"* Concrete International, February, pp. 53-56.