The University of Texas at Austin

Departmental Report

Title: "Bond Strength of Two-Bar Bundles in Two Layers with Epoxy Coating

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DEDICATION

To my wife and my daughter as well as to my parents, brothers and sisters for their love and support during my studies.

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On a personal level, I am grateful to my wife Nohemi “Mimi” Elizondo and my daughter Priscila for their love, patience and support; and for all the time that I could not spend with them.

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VITA

Rubén Elizondo Ochoa was born in Monterrey N. L., México, on August 27, 1965 to Carlos Elizondo and Mireya C. Ochoa. He graduate from the Universidad Autonoma de Nuevo León on December 1987 and got his Bachelor of Civil Engineering in 1991. After working for two years as a professional engineer, he returned to the UANL to continue with his studies. After being studying the Master Degree program at the UANL he was accepted in the Graduate School of the University of Texas at Austin.

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CHAPTER 1

INTRODUCTION

1.1.- General Background

Corrosion of steel reinforcement is one of the main causes of deterioration in concrete structures exposed to a severe environment. Some examples include bridge decks and parking garages where deicing salts are used; chemical plants, sewage treatment plants, refineries and power plants where chemical products are utilized. Concrete structures in coastal regions exposed to sea water or sea spray often exhibit rapid deterioration.

In order to prevent corrosion, epoxy-coated bars have been used in many of these structures. Epoxy-coating is applied using a fusion-bonded process to coat the steel surface and to provide an effective and economical method of protection against corrosion. However, research has shown that the use of epoxy-coating affects bond between steel and concrete. Development and splice lengths must be increased when using epoxy-coating.

Bond in epoxy-coated bars can be further complicated when the design requires a great deal of steel in members and the section size must be limited. A common practice to solve this problem is to place the bars in bundles and even, in some cases, to use several layers of bars. Although building codes provide some guidance on anchorage, development and splice lengths on epoxy-coated and bundled bars there is no experimental support for cases involving both bundling and coating.

Experimental tests of epoxy-coated bars in bundles are required in order to provide data for checking or modifying code provisions.

1.2.- Project Background

This report is part of Project 1363 on anchorage and development length of groups of reinforcing bars sponsored by the Texas Deportment of Transportation (TxDOT). A thesis on "Anchorage and Development of Two-Bar Bundles in One and Two Layers" by Chen(1) and one on "Bond and Development of Bundled Reinforcing Steel" by Grant(2) summarize previous work done on this project. The tests reported are on models of a typical TxDOT application. The reinforcement for inverse T-beams or pier bent caps is often designed with two bar bundles in one or two layers. Figure 1.1 shows a pier bent cap, while Figure 1.2 shows a typical reinforcement cage.

1.3..- Object and Scope

The objective of this report is to compare the bond strength of two bar bundles in two layers with and without epoxy-coating. In addition, the influence of casting position in both cases was considered. The results were also related to existing codes and compared to Chen's work and to other research on single epoxy-coated bars in one layer.

Four tests were conducted using one beam (with four anchorage regions) in which the only variables studied were the presence of the epoxy-coating and the casting posistion. Other variables such as concrete strength, diameter of the bar, steel yield stress, anchorage length, face cover, and clear spacing were kept constant.

Figure 1.1.- Pier Bent Cap

Figure 1.2.- Cage Reinforcement of a Bent Cap

CHAPTER 2

Review of Previous Research

2.1.- Review on Bond

Most of the literature dealing with bond strength is based on tests of single uncoated bars in one layer. From all these tests, it has been found that bond strength depends on the diameter of the bar, face and side concrete cover, clear spacing between bars, transverse reinforcement, concrete strength, embedded length, and casting position. Based on over 500 available tests on bond, Orangun et al(3), derived an empirical equation using a nonlinear regression analysis. The bond strength of an anchored deformed bar or a splice is a function of the depth of cover, spacing between adjacent bars or splices and the amount of transverse reinforcement. This formula considers the total bond strength as a combination of the bond due to concrete around the bar and that due to transverse reinforcement confining the bar:

$$\mathbf{u = u_c + u_{tr}} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad \textbf{Eq. 2.1}$$

where u = total bond stress, and u_c and u_{tr} represent the contribution to bond due to the concrete and to the transverse reinforcement.

The average bond stress contributed by the concrete u_c can be expressed in a non-dimensional way by the parameters $\frac{u_c}{\sqrt{f'c}}$, $\frac{c}{d_b}$, $\frac{d_b}{l_s}$, as:

$$\frac{u_c}{\sqrt{f'c}} = 1.2 + 3\frac{c}{d_b} + 50\frac{d_b}{l_s} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad \textbf{Eq. 2. 2}$$

The increment in bond stress due to the transverse reinforcement can be expressed by the factor K_{tr} as follows:

$$K_{tr} = \frac{u_{tr}}{\sqrt{f'c}} = \frac{A_{tr} f_{yt}}{500 s d_b} \quad \text{. . . . .} \quad \textbf{Eq. 2. 3}$$

The total bond stress as expressed by the Orangun formula is as follows:

$$u = \left[1.2 + 3\frac{c}{d_b} + 50\frac{d_b}{l_s} + K_{tr}\right]\sqrt{f'c} \quad \text{. . . . .} \quad \textbf{Eq. 2. 4}$$

where $K_{tr} = \frac{A_{tr} f_{yt}}{500 s d_b}$ and :

u = ultimate bond stress, psi

c = the minimum of the concrete cover and half the clear spacing of adjacent bars, in.

d_b = bar diameter, in.

l_s = embedded length, in.

K_{tr} = bond stress factor due to the transverse reinforcement

f_{yt} = yield strength of the transverse reinforcement, psi.

s = spacing of transverse reinforcement, psi.

f_c' = compressive concrete strength, psi

A_{tr} = area of transverse reinforcement crossed by the splitting plane at a single anchored reinforcing bar, in^2.

= $A_{tr} = \frac{\sum a_b}{n_s}$, where a_b is the area of transverse reinforcement per leg and n_s is the number of bars enclosed in the cross section. See Figure 2.1.

Equation (2.4) above has some limitations. As c/d_b increases the bond strength increases and for large c/d_b ratios, direct pullout could occur. Test data indicated that

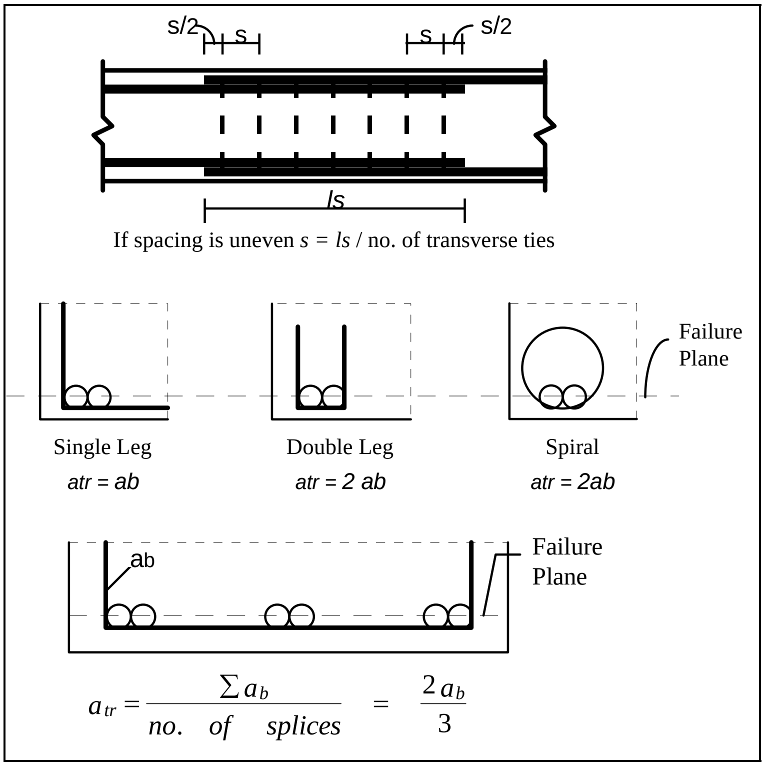

Figure 2.1.- Definition of transverse reinforcement, A_{tr}, by Orangun et al.,[3]

for a c/d_b ratio of 2.5 or more, strength did not increase. Also, it was found that large amounts of transverse reinforcement become ineffective since a splitting failure mode is no longer produced. To reflect this observation, K_{tr} was limited to 3. It was also observed that bond strength was affected by the casting position of the bar. In the relatively few tests with top bars, bond strength was about 82 to 88 % of that for bottom bars. However, as there were very few tests with top bars, it was recommended that for top bars, the development length be multiplied by 1.3. The last observation, based on the available data, was that the empirical equation gives better results when the factor $c_s/(c_b\ d_b)$ is less than 3. For values between 3 and 6 a reduction factor in the splice or development length of 0.9 was proposed as well as a factor of 0.7 for ratios higher than 6.

If bond stress is assumed to be uniform along the bar, the average bond stress can be obtained by setting the total bond force around the bar equal to the tensile force in the bar. As seen in Figure 2.2, the total bond force equals the average bond stress (u) times the area of the perimeter of the bar ($\pi\ d_b$) times its anchorage length

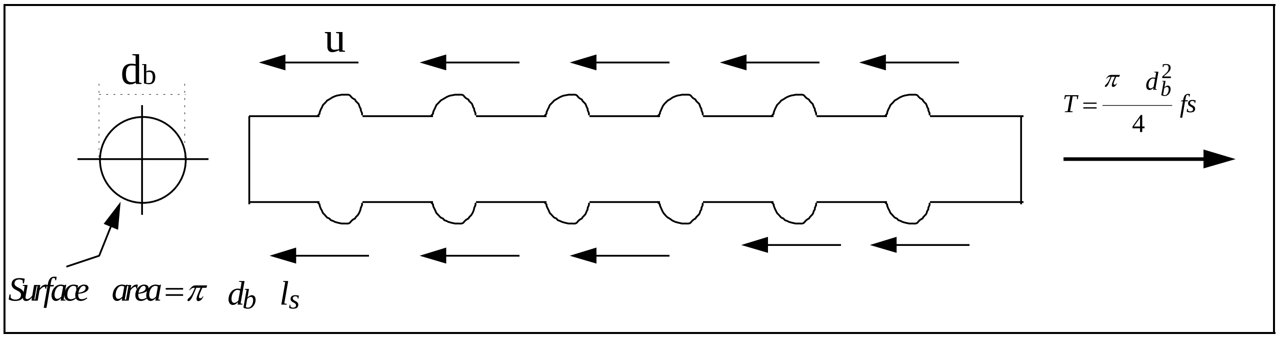

Figure 2.2.- Average bond stress along bar.

(ls), and the tension force in the bar is equal to the area of the bar ($\pi\ d_b^2/4$) times the stress on the bar. In equation form:

$$l_s \pi d_b u = \frac{\pi d_b^2}{4} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \qquad \textbf{Eq. 2. 5}$$

And rearranging Eq. 2.5

$$u = \frac{f_s d_b}{4 l_s} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \qquad \textbf{Eq. 2.6}$$

Combining Eq. 2.6 with Orangun's formula (Eq. 2.4) and solving for the length :

$$l_s = \frac{d_b \dfrac{f_s}{4\sqrt{f'c}} - 50}{1.2 + 3\dfrac{c}{d_b} + \dfrac{A_{tr} f_{yt}}{500 s d_b}} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \qquad \textbf{Eq. 2. 7}$$

2.2.- Previous Research on Bundled Bars

Prior to the present research program, very little data on bundled bar behavior was available. In 1958, Hanson and Reiffenstuhl [(4)] reported their results on tests of pairs of beam specimens with conventionally spaced and bundled longitudinal reinforcing bars. Each set of beams had the same number and type of bars. The tests included groups of four No. 6, four No. 8 or three No. 9 bars in different configurations. Comparisons with respect to flexural cracks, steel stress distributions and ultimate strength were made. However, no significant difference in behavior or ultimate strength was found.

2.3.- Previous Research on Bond on Epoxy-Coated Bars[5]

2.3.1.- National Bureau of Standards Tests.

In 1976, Mathey and Clifton[6] reported the first study on bond of epoxy-coated bars done at the National Bureau of Standards. Bond strength of epoxy-coated bars was compared with uncoated bars in pullout tests. The reinforcing steel used was all No. 6, Grade 60 bars. Twenty-three coated bars with varying coating thickness ranging from 1 to 11 mils, and two bars with a coating thickness of 25 mils were used. The coated bars were compared to five uncoated bars. The variables studied were: coating thickness, deformation pattern, and the method for coating application.

It was found that the average value of the applied load corresponding to the critical bond strength in bars having epoxy coating thickness between 1 and 11 mils was only 6% less than the average load applied to uncoated bars. Based on this critical bond strength it was concluded that bars with coating 1 to 11 mils thick develop acceptable bond strength. In these tests the critical bond strength was considered as the lesser of the bond stress corresponding to a loaded-end slip of 0.01 in. or that corresponding to a free-end slip of 0.002 in. However, the critical bond computed in this way does not give the actual bond strenght of the bar.

From all the tests, only the two bars having the 25 mil coating thickness had a bond failure. All the other coated bars with 1 to 11 mil coating thickness, as well as the uncoated bars, yielded in the tests. Based on this, it was recommended that bars with coating thickness greater than 10 mils not be used.

Since most of the bars yielded during the tests, there is no information on the steel stresses at failure and the actual bond strength capable of being developed can not be determined. As stated before, if the embedded length and the cover are big enough, any bar with or without coating can develop yield.

2.3.2.- North Carolina State University.

In August 1982, Johnston and Zia[7] conducted another study at North Carolina State University. In these case, three slab specimens with uncoated No. 6 bars and three with coated No. 6 bars were used to compare strength, crack width and crack spacing with. Also, forty beam end specimens with No. 6 or No. 11 bars with three different embedment lengths were used to compare strength under static and fatigue loading. Steel grade and production heat, concrete mix, and epoxy coating thickness were kept constant.

In order to simplify the measurement of cracks, the slabs were tested as simply supported beams with the tensile surface at the top.

Little difference in crack widths and spacing, deflections and the ultimate strength were found between coated and uncoated specimens. The failure load of the epoxy-coated bar specimens was about 96% of that of the uncoated specimens. Because of the test setup and the large development length used, the tests resulted in flexural failure and the actual bond strength could not be measured.

Since the beam end specimens were flexural type specimens, the loads were applied directly to the reinforcing bar. Transverse reinforcement was provided for these specimens. In this case, the loading was concluded either when the bar reached 125% to 140% of the yield stress (long embedded lengths) or upon pullout (short embedded lengths).

Bond splitting cracks and flexural cracking were developed in epoxy-coated bars at lower load levels than for specimens with uncoated bars. Also, at the same level of stress, the epoxy coated bar specimens recorded larger slips. It was found that changing the embedment length or the bar size from No. 6 to No. 11 did not influence the performance of the epoxy-coated bar specimens relative to the uncoated bar specimens.

Based on tests that had a pullout bond failure, the epoxy coated bars developed about 85% of the bond strength of the uncoated bars. Similar results were found in fatigue and static tests. It was recommended that when using bars with epoxy coating the development be increased by 15% in order to account for the reduction.

2.3.3.- Exploratory Studies at The University of Texas.

In 1987, Treece[(8)] tested twenty-one beam specimens to determine the influence of epoxy coating on bond strength, member stiffness and on the spacing and width of cracks. The variables were bar size, concrete strength, casting position and coating thickness. All of the same sized bars were from the same heat of steel and no transverse reinforcement was provided in the splice region.

Different combinations of the variables were examined in several series. In each series, a control specimen with uncoated bars and a specimen with bars having a 12 mils coating were included. Since a minimum of 5 mils and a maximum of 12 mils are specified by ASTM A775/A 775M-88a, in some series a third specimen with a 5 mil coating was added. Most of the specimens were cast with bars in the top position and some were bottom cast.

All tests resulted in a splitting failure at the splice region. Test results showed that only 67% of the bond strength of the uncoated bars is developed in the epoxy-coated bars with an average thickness above 5 mils. This reduction was consistent for all the variables studied. The only variable affecting the bond strength in companion specimens was the presence of the epoxy coating.

Little difference in flexural behavior was noted between specimens with and without epoxy coating. It was also found that the specimen with epoxy coating had fewer, but wider cracks than the uncoated specimen.

Based on the test results, Treece recommended a 50% increase in the basic development length where the concrete cover is less than 3 d_b or the bar spacing is less than 6 d_b. Moreover, based on Johnston and Zia's test results, it was also recommended to increase the basic development length by 15% for all other cases where epoxy coating is used. It was also suggested that the combination of factors for top reinforcement and epoxy coating be limited to 1.7.

The design recommendations made by Treece were later adopted by ACI 318 in the 1989 Building Code(9) with the only modification being an increase of 15% rather than 20% as originally suggested. Since tests did not consider the effect of transverse reinforcement, it was also indicated that more research in this area must be done.

2.3.4.- Purdue University.

In 1989, Cleary and Ramirez(10) reported the results of an experimental program conducted to evaluate the bond strength of epoxy-coated splices in constant moment regions of slab specimens. The influence of epoxy coating on member stiffness and on the spacing and width of cracks was also studied.

Four slab specimens reinforced with epoxy-coated bars and four companion slabs with uncoated bars were tested. All the steel bars were from the same heat and the average coating thickness was 9.0 mils. No transverse reinforcement was used. Different concrete strengths (4 ksi and 8 ksi) and embedded lengths were used.

In two of the four uncoated slab specimens the steel bars yielded, and the other two resulted in bond splitting failure. Based on the two specimens that resulted in bond failure, the corresponding specimens with epoxy-coated bars developed 97% and 65% of the bond strength. The former ratio was obtained from a specimen with 12 in. splices and 4 ksi nominal concrete strength. The latter was obtained with 10 in. splice and 8 ksi nominal concrete strength. The large difference was attributed to the concrete strength and to the number of flexural cracks.

As in Treece's tests, it was also noted that there was no loss of slab stiffness due to the epoxy coating and that there were fewer cracks but they were wider in specimens with epoxy-coated bars. Cleary and Ramirez also concluded that there appeared to be no significant difference in the behavior of beams and slabs with epoxy coating designed to fail in a splitting mode of failure.

Based on only two slab specimens with high strength concrete (8200 psi), they concluded that there is a need in the design provisions to account for the effect of concrete strength on the reduction of bond strength when epoxy-coated bars are used.

2.3.5.- University of California at Berkeley.

In 1989, DeVries and Moehle[11] conducted an experimental program to examine the effects of concrete strength, casting position, epoxy coating, and the presence of an anti-bleeding agent on the bond strength of splices. Three nominal concrete strengths of 8, 10, and 15 ksi were tested. The reinforcing steel bars were Grade 60, No. 6 and No. 9 bars. All the bars of the same size came from the same heat. The nominal thickness of the epoxy coating was 8 mils. Some of the specimens had transverse reinforcement along the splice region.

All the beams tested had a sudden but not explosive splitting failure mode. The tests showed that bond strength was affected by the casting position and the presence of epoxy coating in the bars. However, it was observed that the effects were not cumulative. Devries and Moehle concluded that the modification for top to bottom cast bars given in Section 12.2.4.3 of the 1989 ACI Code (ACI 318-89), was not needed. The tests results also showed that the bond strength of a splice in either top or bottom cast bars position is not significantly altered by the anti-bleeding agent.

Based on their observations, Devries and Moehle suggested the use of the design equation proposed by the ACI Committee 408[12] : $l_s = \dfrac{5500\, A_b}{\phi\; K\sqrt{f'c}}$, with a modification factor of 1.3 for top bars and for epoxy-coated bars regardless of the casting position. Their recommended design approach was also conservative for specimens with concrete strength of 16,100 psi, which is close to the practical limit for concrete strength. Therefore, Devries and Moehle conclude that the upper limit on the value $\sqrt{f'c}$ in the recommended design equation as is done by the ACI Code (318-89) development length specifications was not needed. Since all the test specimens had transverse reinforcement in the splice region and no companion beams without transverse reinforcement were tested, no conclusions could be drawn on the effect of transverse reinforcement on the bond of epoxy-coated bars.

2.3.6.- The University of Texas at Austin.

In 1990, two experimental programs were conducted at the University of Texas. In the first study, Hamad et al[5] reported a test program in which the fundamental bond properties of epoxy-coated bars were examined. The variables studied were the bar size, the coating thickness, the bar deformation pattern, the rib face angle, the degree of confinement of the anchored bar and the concrete strength. Eighty specimens with several combinations of the variables were tested. In this case, the pullout tests without transverse reinforcement included different confinement load.

The reinforcing steel consisted of Grade 60, No. 6 and No. 11 bars. Bars of the same size were from the same heat of steel. In some series, machined bars with different deformation pattern were used. For these bars, hot rolled Grade 1045 carbon steel was used. The epoxy-coated bars tested had an average measured epoxy coating thickness of 5 mils, 8 mils, and 12 mils. The nominal concrete strength was 4000 psi and 8000 psi.

The following observations were made in specimens without transverse reinforcement:

1.- The ultimate bond strength in a reinforcing bar, uncoated or epoxy-coated, increased as the confining load increased. The largest range of epoxy-coated to uncoated bond ratio was for the lowest level of confining load. This range varied form 0.65 to 1.00. At higher levels of confining load, the range became smaller (0.79-0.92).

2.- The bar size did not influence the reduction in bond strength of epoxy-coated bars relative to uncoated bars.

3.- The reduction in bond strength and in load-slip stiffness of epoxy-coated bars relative to uncoated bars is independent of the epoxy thickness and deformation pattern.

4.- The ultimate bond strength in a reinforcing bar, uncoated or epoxy-coated, increased as the concrete strength increased. However, the results showed no significant difference in the bond ratio of epoxy-coated bars relative to uncoated bars as the concrete strength increased.

The conclusions from specimens with uncoated transverse reinforcement are as follows:

1.- The bond strength ratios, epoxy-coated to uncoated bars, varied from 0.72 to 0.79.

2.- The bond strength increased in both epoxy-coated and uncoated bars as the number of ties increased. However, the increase in transverse reinforcement did not affect the bond strength of epoxy-coated bars relative to uncoated bars.

In the second study, Hamad et al.[(5)] tested twelve beams to determine the effect of coated transverse reinforcement on the bond strength of epoxy-coated bar splices. All the specimens had bars only in a top cast position. A nominal concrete strength of 4,000 psi was used. The reinforcing steel was Grade 60, No. 6 and No. 11 bars. The nominal coating thickness on the longitudinal steel was 8 mils while on the transverse reinforcement, the measured thickness was 9 mils.

Again, it was found that the epoxy-coated specimens had wider flexural cracks at larger spacings than with uncoated bars. However, it was noted that the total width of all cracks in both type of specimens (with epoxy-coated and uncoated bars) was about the same. Also, there was little difference in stiffness between the epoxy-coated and uncoated specimens with no transverse reinforcement. However, in the specimens with transverse reinforcement, the stiffness in the epoxy-coated specimen showed a gradual decrease relative to the companion uncoated specimen as the load approached failure and as the amount of transverse reinforcement in the splice region increased. Regardless of this decrease, it was shown that the epoxy coating did not significantly affect the flexural cracking load.

The results showed a relative bond strength ratio of coated to uncoated bars in specimens without transverse reinforcement of 0.74 for No. 11 bars and 0.67 for No. 6 bars. However, the bond capacity improved with the increase of transverse reinforcement. This improvement was greater for the epoxy-coated bar specimens. For uncoated No. 11 bars, the bond strength increased 8 % using K_{tr} = 1.02, and 15 % with K_{tr} = 2.04. On the other hand, for coated No. 11 bars, the bond strength increased 19 % with K_{tr} = 1.02, and 31 % with K_{tr} = 2.04. In the case of No. 6 bars, the increase in specimens without transverse reinforcement was 10 % with K_{tr} = 1.02, while for specimens with transverse reinforcement, it was 22%.

It was concluded that this increase was independent of the number of splices, bar size or bar spacing. Also, it was found that the average bond ratio for beams with ties in the splice region was 0.81.

Based in their results and in the previous references, Hamad et. al. suggested some modifications to current Code provitions.

2.4.- Current ACI 318-89 Code Provisions and 1992 AASHTO Specifications.

2.4.1.- Specifications on bond and bond in bundles.

Both the ACI code and the AASHTO Specifications use the concept of development length, which is the shortest length bar required for the stress to increase along the bar from zero at the load to yield stress f_y. The development length l_d is computed as the product of the basic development length, l_{db}, and the applicable modification factors that affects the bond. The development length for No. 11 and smaller bars, in both provisions, is:

$$l_{db} = \frac{0.04 A_b f_y}{\sqrt{f'c}} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad \textbf{Eq. 2.8}$$

The factors affecting the development length and their values in both provisions are not all the same. In the ACI code, the factors accounting for clear spacing, cover, and transverse reinforcement may increase the development length while these factors in the AASHTO will only reduce the development length if they apply. Reductions for excess reinforcement, wide bar spacing and cover, and close pitch in transverse spiral reinforcement are applied in both provisions. Also, both provisions specify increase for casting position, lightweight aggregate, and epoxy coating.

With respect to bundled bars, section 12.4 of the ACI code and section 8.28 of the AASHTO specifications state:

> The development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased by 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.

However, there is no special requirement about two-bar bundles because it is considered that the surface bonding area remains the same for the case of two-bar bundle.

2.4.2.- Specifications on bond for epoxy-coated reinforcement.

Section 12.2.4.3 of the 1989 ACI code specifies that for epoxy-coated bars with cover less than 3 d_b or with clear spacing between bars of 6 $d_{b,}$ the development length should be multiplied by 1.5, and for all other conditions should be multiplied by 1.2. It is also stated that the product of the factors for top casting and for epoxy coating needs no to be taken greater than 1.7.

The 1992 AASHTO Specifications(13) use the same criteria, however, the 1.2 factor for all other cases is taken as 1.15 as suggested by Treece.

Chapter 3

Test Description

3.1.- Design of Specimen

3.1.1.- Introduction

In this phase of Project 1265, the effect of epoxy coating on bond in two-bar bundles in two-layers is examined. Tests were conducted using a beam specimen similar to that used in previous phases of the project. The variables studied included the presence of epoxy coating on the bar and the casting position. All other variables were kept constant. The test region, where the bars were anchored, was a zone of zero shear.

To minimize the number of specimens to cast, only one beam element with four separate test regions was built. In one end of the beam (two test regions), uncoated reinforcing bars were used; however, in the other end (two test regions), both horizontal bars and transverse reinforcement were painted with patching material used to repair damage to epoxy coated bars. The layout of the test regions permits comparison of identical bars with only the coating and casting position (top or bottom) varied. Further, the two tests with uncoated bars can be compared with Chen's results(1) of tests with similar geometry and layout.

Although paint, rather than fusion-bonded epoxy coating was used, the behavior is assumed to be similar with regard to loss of adhesion between the bar and the concrete. Moreover, all bars of each size were from the same heat of steel. This insured that the bars (coated or uncoated) have identical deformations and mechanical properties.

As in previous tests, the distribution of the steel and the dimensions of the specimen were based on a typical TxDOT bent cap. However, because of the geometry and force limitation of the test floor of the Ferguson Structural Laboratory, a half scale specimen was used.

3.1.2.- Specimen Details

The dimensions of the beam specimen were 22 in. (560 mm) wide by 30 in. (760 mm) deep. Two layers with five two-bar bundles each were used. The steel was placed to provide a face cover of 1 in. (25 mm), a clear spacing between bundles of 2 5/8 in. (67 mm), a side cover of 2 in. (51 mm), and a clear space between layers of 1.25 in. (32 mm). The geometry was the same for top and bottom bar position making a direct relationship between top and bottom casting positions possible.

Because it is common practice, it was decided to use transverse reinforcement in the development length test region even though there is no shear in this zone. The transverse reinforcement consisted of two No. 4 (13 mm) stirrups spaced 5 $^1/_3$ in. (135 mm) the test zone and 5 in. (127 mm) along the beam. Some intermediate horizontal shear reinforcement, two No. 6 (19 mm.) bars on each side, was placed along the length of the beam as shown in Figure 3.1. This additional longitudinal steel was shown by Chen(1) to limit cracking in the test region when a set of bars was loaded to failure.

The anchorage length was designed so that the bars would not reach yield. If the bars fail in bond and if the stresses in the bars are known, it is possible, by linear extrapolation, to determine the minimum required length for the bars to reach yielding. Based on ACI equations for development length and on the empirical equation established by Orangun, et. al.(3), (see Eqs. 2.4 and 2.7), it was decided to test an anchorage length of 16 in. (406 mm). This length is slighter lower than that required for uncoated bars in the bottom cast position and, therefore, it was expected that the four sets of bars would fail in bond. A 16 in. (406 mm) anchorage length was also used by Chen(1) and Grant(2) and a direct comparison could be made with the results from different test series.

The load to be applied to the specimen during the test can introduce confinement forces that could result in higher bond stresses at failure. In order to apply load without the transmission of confinement forces, the section to be tested was isolated. First, the development length to be tested was defined by placing the bar bundles outside of the test zone in light gage steel ducts which extended to the point of maximum moment. In this way, only the length to be tested **(**16 in. (406 mm)**)** and the middle of the beam **(**53 in. (1346 mm)**)** are bonded and will transfer stresses to the

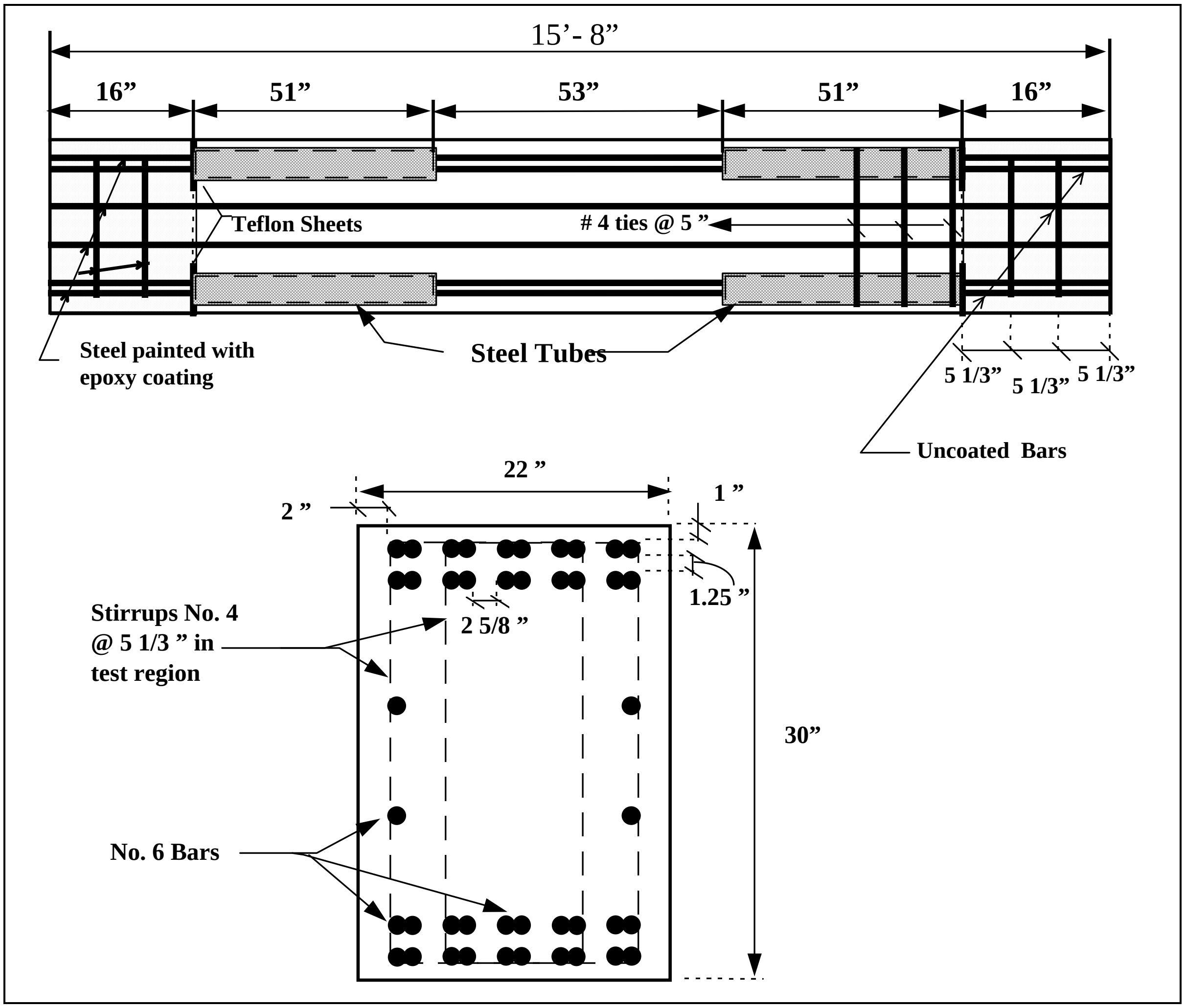

Figure 3.1.- Beam Specimen Cross-Section and Layout

concrete. As the length to be tested is shorter than the bonded length in the middle of the beam, the test region will fail first.

Second, a Teflon sheet was placed between the test region and the unbonded region as a "bond breaker". This "bond breaker" will isolate the test section without introducing a discontinuity between the embedment block and the adjacent concrete. Moreover, as the Teflon sheet surface is very smooth, it will not bond to the concrete and will not transfer the confinement forces to the test zone.

The four tests are referred to by a two word code which identifies the casting position, top or bottom, and epoxy-coated is referred to as "epoxy" while the uncoated condition is referred to as "black". Therefore, top epoxy indicates a test with top cast, epoxy-coated bars. Also, the term outer layer refers to the layer closer to the top or bottom surface of the section.

3.2.- Materials

3.2.1.- Concrete.-

The concrete was ordered from a local ready-mix plant. It was a non air-entrained mix and because of the congestion of the steel the maximum aggregate size was 3/8" (10 mm). The nominal concrete compression strength was 3500 psi. (24 Mpa) at 28 days. Figure 3.2 shows the average compression strength of cylinders at 7, 14, 21 and 28 days respectively. The average compression strength at 28 days was 3620 psi. (25 MPa). Since tests were done shortly after the concrete was 28 days old, it was considered that strength was the same for all four tests.

3.2.2.- Reinforcing Steel.

Bars of each size were from the same heat of steel to ensure identical deformations and mechanical properties. The bond strength was measured using No. 6 (19 mm) longitudinal bars and stirrups made of No. 4 (13 mm) bars. All the steel was Grade 60. Coupons of each bar size were tested and exhibited a well-defined

Figure 3.2.- Cylinder Compression Strength-Age Curve

yield plateau. The average yield stress was 61.4 ksi. (423.32 Mpa) for No. 6 (19 mm) bars and 64.7 ksi (446.1 Mpa) for No. 4 bars.

Epoxy patching material was applied to the No. 6 (19 mm) bars on one end of the beam. The painted length was about 20 in. (508 mm), but only a 16 in. (406 mm) length was used in the test region.. Also, the stirrups to be placed in this region were painted. The coating thickness of the epoxy-coated bars was measured using a Micro-test thickness gage. Three measurements were taken along each longitudinal bar and four in each stirrup (one on each leg). Figure 3.3 and Figure 3.4 show the coating thickness gage and the measurement procedure. The average coating thickness for all the epoxy-coated bars was 4.74 mils with a standard deviation of 1.08 mils. Figure 3.5 shows the distribution of the average coating thickness measured.

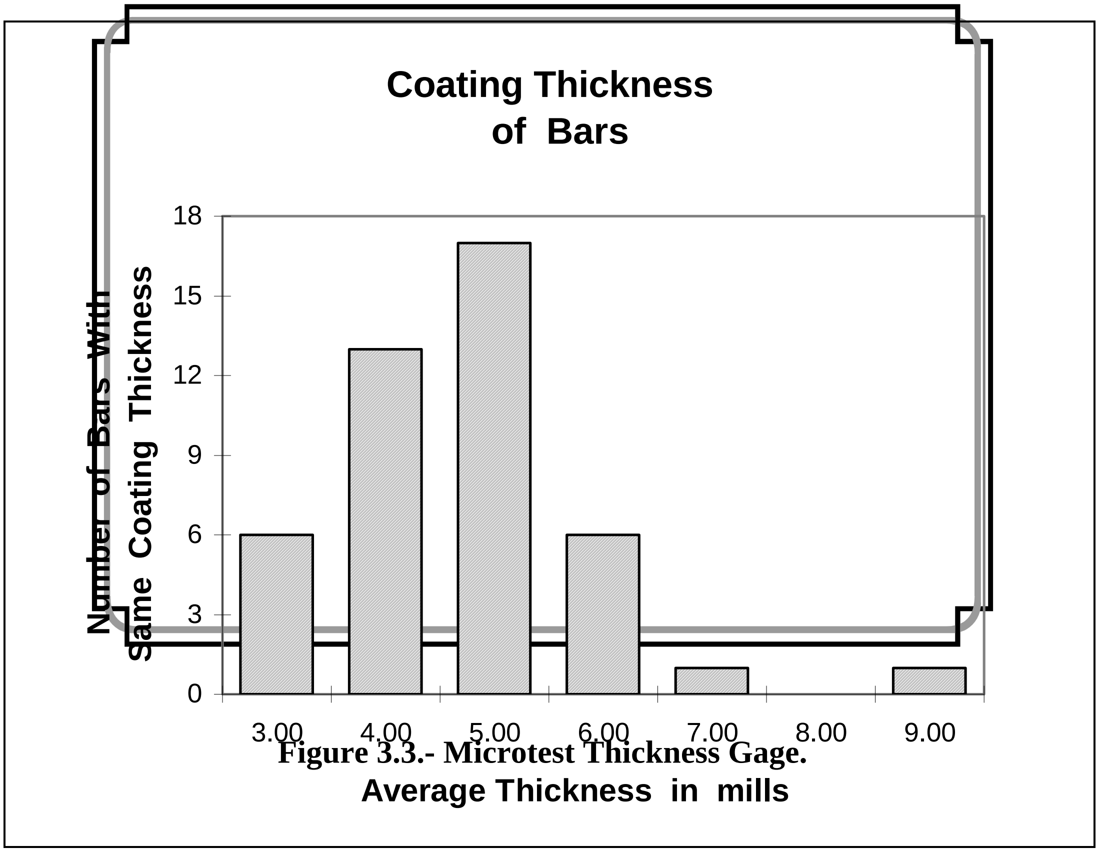

Figure 3.3.- Microtest Thickness Gage.

Figure 3.5.- Distribution of Measured Coating Thickness in Bars

Figure 3.4.- Measuring the Coating Thickness.

3.3.-Construction of the Specimen.

After the steel bars were cut to length, strain gages were placed on the longitudinal reinforcing bars and on the transverse ties. The gages on the longitudinal bars were placed close to the lead and of the anchorage bars, that is slightly over 16 in. (406 mm) from the end. Only three of the five two-bar bundles in each layer were gaged since previous tests had shown that the distribution of stresses in the bars across the section is almost symmetric and there is little loss in accuracy assuming a symmetric stress distribution. On the transverse reinforcement, the gages were placed so they coincided with the expected splitting crack, that is, in the plane of the interior layers of bars.

With the strain gages placed, the longitudinal bars and some stirrups were painted. Since bond strength and not corrosion were of interest, the surface of the bars was cleaned only with a steel brush and acetone. After the coating was applied, the epoxy coating thickness was measured.

Once the bars and the stirrups were gaged, steel tubes were placed on the bars. One tube on each side covered or sheathed two bars. The two bars were inserted into the tubes until only the test lengths **(**16 in. (406 mm)**)** protruded. Silicon caulk was used to seal the gaps between the bars and the tubes in order to prevent cement paste from entering into the tubes.

Using the two bar bundles and the stirrups, the cage was fabricated. First, the top outer layer of bundles were placed and the stirrups, along the beam and out of the test zone, were fixed in their place. Then, the bottom outer layer of bundled bars was inserted and fixed in place. Diagonal bars were used to stabilize the cage and did not extend into the test region, therefore they do not affect any result. After this, the inner layer was inserted and tied. Some temporary guides or supports were used to hold the inner layers in place while they were tied.

When the cage was formed, the “bond breaker” (the Teflon sheet) was placed. The height of the Teflon sheet was centered with respect to the two layers of bundles bars, that is, about 5 in.(127 mm). Holes were drilled in the Teflon sheets so the sheets could slide along the bars until touching the tubes. Again, some silicon caulk was used to seal any gap around the bars and the Teflon sheet.

After the bond breaker was installed, the rest of the stirrups were placed in the test zones. The whole cage was lifted and placed in the form. A picture of the steel cage is shown in Figure 3.6.

The concrete was placed in the form using a bucket transported by the overhead crane. It was placed in 3 or 4 lifts of about 8 (203 mm) to 12 inches (305 mm) and each lift was vibrated. At the same time the specimen was being cast, the cylinder molds for compression strength tests were also being prepared. Figure 3.7 and Figure 3.8 show the concrete placement and cylinder preparation.

Figure 3.6.- Steel Cage of the Specimen

Figure 3.7.- Concrete Placement.

After placing the concrete, the surface of the specimen was finished with trowels. Immediately after finishing, the specimen and the cylinders were cured for about seven days under wet burlap, covered with plastic sheets.

3.4.-Test Setup

3.4.1.- Loading System

The load was applied to the beam specimen as in previous phases of the program. The loading method involved an inverted single beam loaded at midspan. Figure 3.9 shows the sideview of the test setup. The test region at the end of the beam was loaded as a cantilever beam.

The load was provided by two 200 kips rams (890 KN) connected to the same pump so the same pressure and therefore same load were applied by each ram. The ram reacted against four rods which transferred the load to the reaction floor. On the other end of the rams, the load was transferred to a steel loading beam with a roller

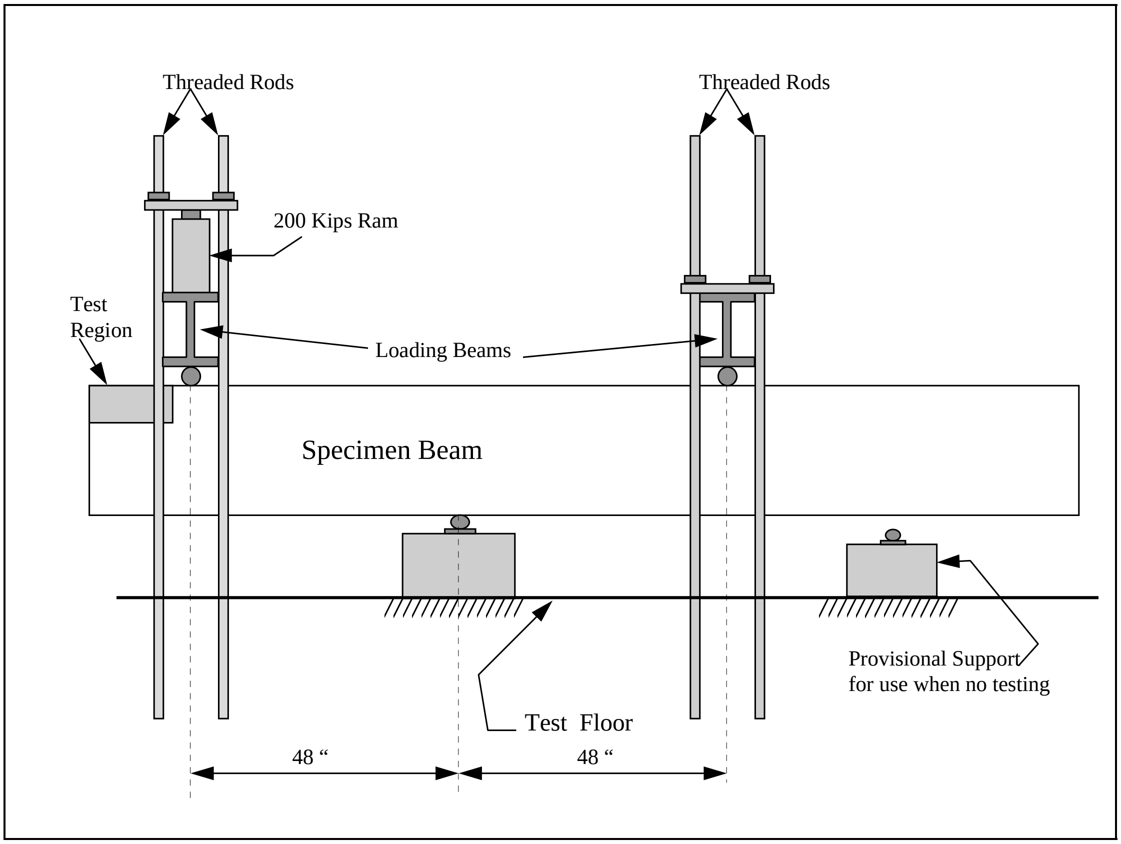

Figure 3.8.- Cylinder Preparation.

Figure 3.9.- Test Setup

support at the base. The roller support allows free end rotation of the beam and distributes the load along the cross section of the specimen. A second loading beam was placed on the other end of the specimen.

When load was applied, the steel bars developed tension stresses that were transferred to the concrete by bond stresses. Bond stress acts only in the test zone and in the middle bond region. As the bonded length is less in the test region, the bond failure will develop there first. The central bonded region provided the necessary resistance for the bars being tested.

A pressure transducer and linear potentiometers were used to measure the load being applied and the displacement of the specimen in the test zone. A picture of the test setup is shown in Figure 3.10.

3.4.2.-Testing Procedure

After setting the specimen in the test setup; strain gages, linear potentiometers and the pressure transducer were connected to a data acquisition system. With any increment of load, the computer was instructed to scan and record the readings on a disk. Also, a print out of each scan with the respective conversion to stresses, load, and deflection in inches was obtained.

The load was manually applied to the specimen with the use of a hydraulic pump. The load was determined by the pressure in the pump and read by a dial gauge. Before the first longitudinal crack appeared in the test region, load increments of 5 to 10 kips (22.24 to 4.45 KN) were applied. After that, load increments were reduced to 2-3 kips (4.90 to 13.34 KN) , to produce a change in the bar stress of approximately 1 ksi (6.89 Mpa) per step until the specimen failed. The time between load increments was restricted by the time required to print data. It was usually between 1 and 2 minutes. The test section was periodically inspected to monitor the existence of cracks and their development.

Figure 3.10.- Test Setup

The test was stopped when the specimen could not take any more load or the concrete cover in the test region split. After the test was completed, photographs were taken to record the failure mode and the crack pattern. Moreover, the concrete cover was removed and the failure plane was inspected. Also, the measurement of concrete cover and spacing between bundles was checked.

Chapter No. 4

Test Results

4.1.- Introduction.

Results of the four tests are presented and analyzed. The behavior of bundled bars is described in terms of the failure mode and crack pattern in the test region. Ultimate bond strength and the effect of casting position are correlated for tests with and without epoxy coating. Also, results of tests with uncoated bars are compared with the corresponding data obtained by Chen(1).

By assuming a linear relationship between bond and development length, it is possible to extrapolate from the test embedded length and compute a minimum length required to reach the yield stress in the bar. This length was compared with that specified by current codes (ACI and AASHTO) and with that suggested by the Orangun's formula.

4.2.- General Behavior.

It was observed that the presence of epoxy coating made no difference in the crack pattern or in the way cracks develop. However, a few more cracks were seen in the top casting position than in the bottom position. Figure 4.1 and Figure 4.2 show the crack pattern in all tests.

First, longitudinal cracks appeared above the bundled bars in both edges of the specimen. Then, a third crack started above the middle bundle and, in the free end, transverse cracks appeared across the inner layer of the bundle bars. As the load increased, transverse cracks appeared directly above the stirrups and, in the free end, a second crack appeared in the outer layer of bars. The specimen failed after the crack in the outer layer was formed. The failures were "side and face split" mode and occurred by crushing of the concrete in the top of the test region.

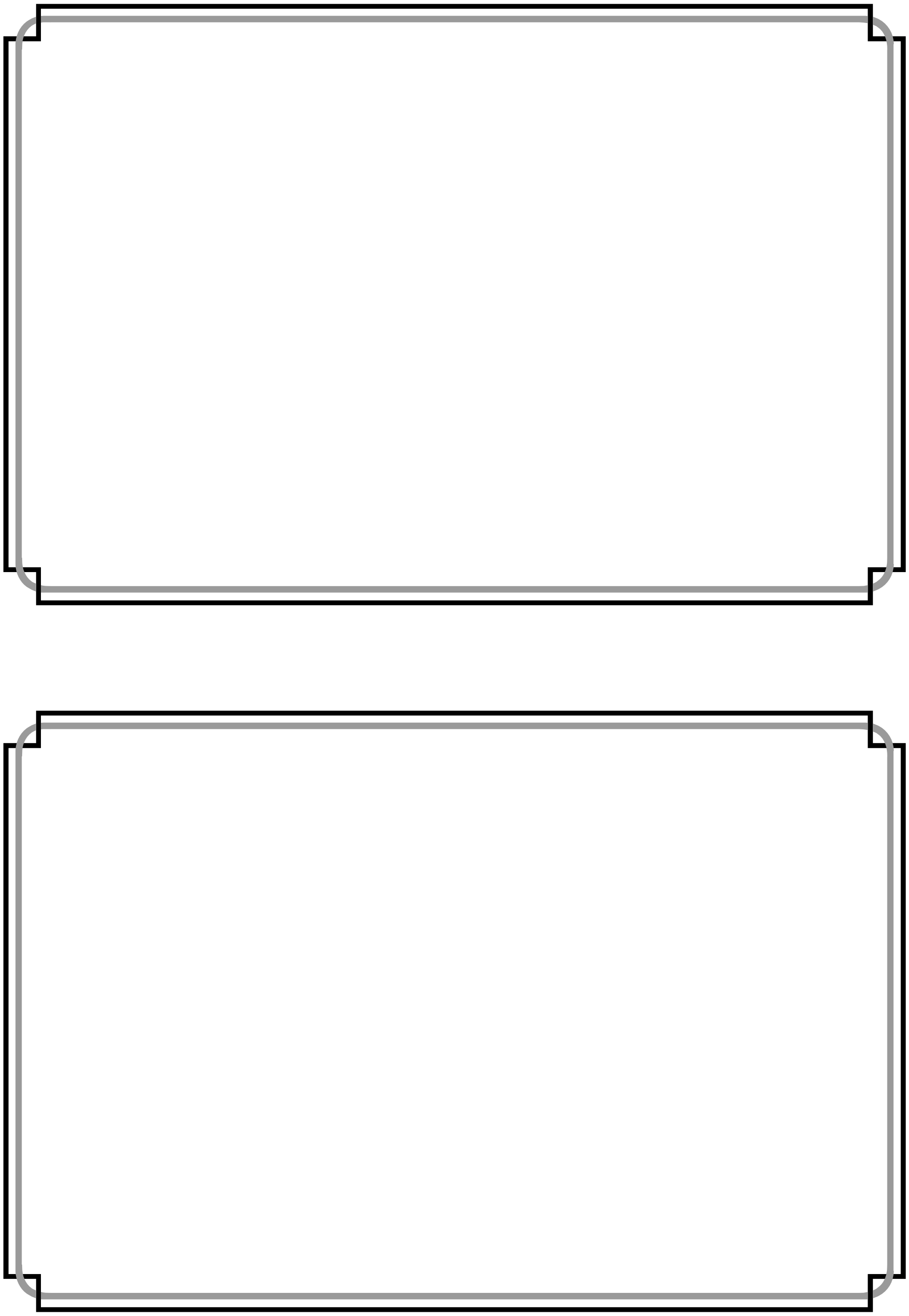

Figure 4.1.- Crack Pattern in Tests with Epoxy Coating.

Figure 4.2.- Crack Pattern in Tests with Uncoated Bars.

After failure, the concrete cover was removed to study the failure plane in the test region. The epoxy-coated bars were clean with no concrete residues on the bar deformations. In some bars, small spots of the epoxy cover peeled off. The concrete cover in contact with the epoxy-coated bars had a smooth glassy surface and the pattern of the bar deformations printed on the concrete were almost intact. There were no signs of concrete being crushed against the bar deformations. Figure 4.3 shows the concrete cover in contact with the epoxy-coated bars. On the other hand, the deformations of the uncoated bars had attached particles of concrete. The concrete surfaces in contact with the bar deformations were rough and signs of concrete crushing against bar deformations was observed.

Table 4.1.- Verification of Accuracy of Strain Gages by Comparing the Applied Moment and Resisting Moments

Test	Applied Moment Ma = 48 x P (Kips-in.)	Resisting Moment Mr = As fs d (Kips-in.)	Error $\% \ error = ABS\left[\frac{Mr - Ma}{Mr}\right]$
TOP EPOXY	10580	11106	4.72 %
BOTTOM EPOXY	10680	10540	1.33 %
TOP BLACK	9625	9995	3.72 %
BOTTOM BLACK	12305	10545	14.3 %

Figure 4.3.- Concrete Cover of Epoxy-Coated Bars After Test.

4.3.- Test Results.

4.3.1.- Verification of Data.

Collected data from tests consisted of steel strains measured by gages placed directly on the reinforcing bars and stirrups, and on load readings from a pressure transducer. Also, beam deflections were measured by linear potentiometers at the free end of the test region. Steel strains were transformed to stresses by using the Young's Modulus of Elasticity (E).

In order to know the reliability of the results, applied moments to the specimen are compared to resisting moments computed from steel stresses. The applied moment is based on the measured external load used and its distance to the support. On the other hand, the resisting moment is determined by an analysis of the cracked section based on elastic behavior and assuming a linear stress-strain diagram. In the analysis, the tension force in each layer of bundled bars was computed assuming symmetry of stresses in the section and was based on the average stress of the bundles bars. During the analysis, the tensile strength of the concrete was neglected. Table 4.1 shows the comparison of applied and resisting moments only at peak load.

The large difference in the case of the bottom black test was studied further. First, the large difference seemed to be caused because some of the bars reached yielding and some of the strain gages were not functioning at the peak load. However, it was found that even for lower loads, where stresses are in the elastic range, the difference still was around 14 %. Then no reason was found that could explain the difference.

4.3.2.- Collected Data.

The parameters and the data measured from each test are shown in Table 4.2. The average steel stresses shown in the table were computed assuming symmetric stress distribution in the section. Embedded lengths, spacing and covers are the actual as-built dimensions. The table also shows a value "c", which will be used for the computation of the bond strength and the development length. This “c” was considered as the average of the minimums of face cover (layer spacing in the case of inner layers), half the clear spacing between bundles, and side cover of all bundled bars. The compression concrete strength f_c’ was 3.62 Ksi (25 MPa), and the yielding strength for reinforcing bars and stirrups was 61.4 Ksi (423 Mpa) and 64.7 Ksi (446 Mpa), respectively.

Table 4.2.- Test Results and As-Built Dimensions.

		TOP EPOXY	TOP BLACK	BOTTOM EPOXY	BOTTOM BLACK
Load at First Crack		167 Kips (743 kN)	160 Kips (714 kN)	150 Kips (669 kN)	180 kips (802 kN)
Maximum load		220 Kips (981 kN)	200 Kips (892 kN)	223 Kips (990 kN)	256 Kips (1140 kN)
Avg. Steel Stresses at Failure	Outer Layer	48.1 Ksi (332 MPa)	46.3 Ksi (320 MPa)	35.4 Ksi (244 MPa)	32.8 Ksi (226 MPa)
	Inner Layer	54.8 Ksi (378 MPa)	40.5 Ksi (280 MPa)	58.7 Ksi (405 MPa)	61.4 Ksi (423 MPa)
Embedded Length		15.75 in. (400 mm)	16 in. (406 mm)	15.375 in. (391 mm)	15.56 in. (395 mm)
Face Cover (Cb)		1.25 in. (32 mm)	1 in. (25 mm)	1 in. (25 mm)	1.25 in. (32 mm)
Avg. Clear Spacing (Cs)	Outer Layer	2.125 in. (54 mm)	2.125 in. (54 mm)	2.125 in. (54 mm)	2 in (51 mm)
	Inner Layer	2.25 in. (57 mm)	2.125 in. (54 mm)	2.375 in. (60 mm)	2 in (51 mm)
Avg. Layer Spacing (Cs)		1.25 in. (32 mm)	1.13 in. (29 mm)	2 in (51 mm)	2 in (51 mm)
Avg. Value for “c”	Outer Layer	1.06 in. (27 mm)	1 in. (25 mm)	1 in. (25 mm)	1 in. (25 mm)
	Inner Layer	1.125 in. (29 mm)	1.06 in. (27 mm)	1.19 in. (30 mm)	1 in. (25 mm)

4.4.-Bond Strength.

4.4.1.- Computation.

The bond strength was based on the average stress along the embedded length. As explained in section 2.1, the average bond strength is computed by dividing the force in the bar by the surface area along the anchorage length. Equation 2.6 shows that:

$$u_t = \frac{u}{\sqrt{f'_c}} = \frac{f_s d_b}{4 l_s}$$

where u_t represents the measured bond strength from tests and normalized with respect to the concrete strength. On the other hand, Orangun's formula was used to compute the theoretical bond strength and to provided a comparative reference between these and other tests reported in the literature. The theoretical bond strength is represented as:

$$u_o = \frac{u}{\sqrt{f'_c}} = \left[1.2 + 3\frac{c}{d_b} + 50\frac{d_b}{l_s} + K_{tr}\right]$$

where u_o represent the average bond strength computed by the Orangun's formula and normalized with respect to the concrete strength. In this equation, the influence of the transverse reinforcement is considered in the term K_{tr}. However, the term A_{tr} used to compute K_{tr} has been modified by Chen(1). For cases of bundled bars in two layers, he defined A_{tr} as the summation of the product of each leg area times the number of splitting planes crossed by the transverse reinforcement, divided by the number of bundled bars which are enclosed by the transverse reinforcement. That is:

$$A_{tr} = \frac{\sum a_{bi} m_i}{n}$$

where a_b is the area of one leg, m is number of splitting planes crossed by the transverse reinforcement and n is the number of bundled bars enclosed by the transverse reinforcement. For this case the term K_{tr} gives a value of 5.2; however, as described in section 2.1, this value is limited to 3.0. Even though a different spacing was used by Chen, he obtained a K_{tr} of 3.5 which was used in his calculations.

Table 4.3 shows the bond strength. For the case of top casting position, the computed bond strengths by the Orangun's formula were divided by a factor of 1.3. No other factor was used in the calculations to account for the epoxy coating. In order to account for the variations in concrete cover, the table includes a modification factor for stresses in the bars. This modification factor was computed as the ratio of bond strength computed for the standard cover of 1 in. (U_o for c = 1 in.) and the bond strength computed for the existing cover (U_o). The modification factor was used to adjust bond strengths used in the comparisons.

Chen's[1] results of tests with two bar bundles in two layers and modified for a K_{tr} value of 3 are also reproduced. These values do not need to be modified for concrete cover since all had a cover of 1 in. (25 mm).

4.4.2.- Bond Efficiency.

The bond efficiency was computed as the measured bond strength (U_t) divided by its theoretical bond strength (U_o). Table 4.3 includes the bond efficiency ratios for the outer and the inner layer, as well as the averages.

As can be seen in the table, the top epoxy test has a higher bond ratio (1.21) than the top black test (1.03) and the bottom epoxy ratio (0.87) is just smaller than the bottom black ratio (0.89). It is also shown that the measured bond strength for the top epoxy test is higher than the others. The top epoxy test represents a case in which the bond strength was higher than normal. It would be expected that if a larger number of tests had been conducted, the average results would not have much large variations. On the other hand, although the average bond efficiency ratio of the bottom black test is similar to that for the bottom epoxy test, and it should be noted that the inner layer of bars yielded.

Table 4.3.- Bond Strength Comparison.

TEST	Ls in.	LAYER	"c"	fs Kips	U_t	U_O	U_t/U_O	U_O for "c" = 1	Mod Factor	fs Mod	U_t Mod
TOP EPOXY	15.75	OUTER	1.063	48.13	9.52	8.33	1.14			46.48	9.197
		INNER	1.125	54.82	10.85	8.52	1.27			52.94	10.476
		AVG.	*1.094*	*51.48*	*10.19*	*8.43*	*1.21*	*8.14*	*0.97*	*49.71*	*9.84*
TOP BLACK	16	OUTER	1	46.34	9.03	8.11	1.11			45.80	8.92
		INNER	1.063	40.54	7.90	8.30	0.95			40.07	7.80
		AVG.	*1.031*	*43.44*	*8.46*	*8.21*	*1.03*	*8.11*	*0.99*	*42.93*	*8.36*
BOTTOM EPOXY	15.375	OUTER	1	35.43	7.18	10.64	0.67			34.22	6.94
		INNER	1.188	58.73	11.90	11.39	1.05			56.73	11.50
		AVG.	*1.094*	*47.08*	*9.54*	*11.01*	*0.87*	*10.64*	*0.97*	*45.48*	*9.22*
BOTTOM BLACK	15.56	OUTER	1	32.83	6.58	10.61	0.62			32.83	6.58
		INNER	1	61.41*	12.30	10.61	1.16			61.41	12.30
		AVG.	*1*	*47.12***	*9.44*	*10.61*	*0.89*	*10.61*	*1.00*	*47.12*	*9.44*

CHEN'S TOP	16	OUTER	1	40.80	9.47	8.11	1.17
		INNER	1	32.70	7.59	8.11	0.94
		AVG.		*36.75*	*8.53*	*8.11*	*1.06*
CHEN'S BOTTOM	16	OUTER	1	47.00	10.19	10.54	0.97
		INNER	1	43.70	9.48	10.54	0.90
		AVG.		*45.35*	*9.84*	*10.54*	*0.93*

* yielding stress   ** computed with yielded bars

4.4.3.- Comparison with Chen's Tests[(1)].

Bond stresses measured in tests without epoxy coating are compared with Chen's tests. The use of the modified bond strength ($U_{t\ modified}$) permits a direct comparison since they are normalized with respect to the concrete cover. For the top casting position of these tests, $U_t = 8.36$, while in Chen's test $U_t = 8.53$, this represents a difference of 2%. For the bottom casting position, $U_t = 9.44$ compared with 9.84 in Chen's test, for a difference of 4 %.

Even though measured bond strengths correlate well, the concrete strength used in tests are different. In any case, the concrete strength used by Chen (2550 psi (18 MPa) and 2920 psi (20 MPa)) is lower than the one used in this specimen (3620 psi (25 MPa)). The peak load found by Chen for the top casting position (219.7 Kips (977 kN)) and for the bottom casting position (256.5 Kips (1140 kN)) are comparable with those obtained in this test program (200.5 Kips (889 kN) and 256.4 Kips (1140 kN) respectively).

Chen compared the stresses in the inner and in the outer layer of bars and found that the stresses in the inner layer were usually close to or higher than those in the outer. However, he found that at the peak load the stresses in the outer layer were higher. Figures showing load versus stresses in bars for Chen’s tests are reproduced in Figure 4.4 and 4.7. Figures 4.5, 4.6, 4.8 and 4.9 show load versus stresses curves for this program. It can be seen that for the top casting position the stresses in the outer layer of bars decrease when the load is close to maximum and stresses in the bars in the inner layer increase. The same trend is seen for the bottom casting position. However, the change in slope ocurred between the first crack and the peak load.

4.4.4.- Influence of Epoxy Coating.

Measured bond strength is compared in tests with and without epoxy coating. As mentioned before, the top epoxy test resulted in an extreme case with higher values than normal. No reason for these high values could be determined. Its modified

Figure 4.4.- Load versus Stresses of Bundled Bars
Test: Chen's Top

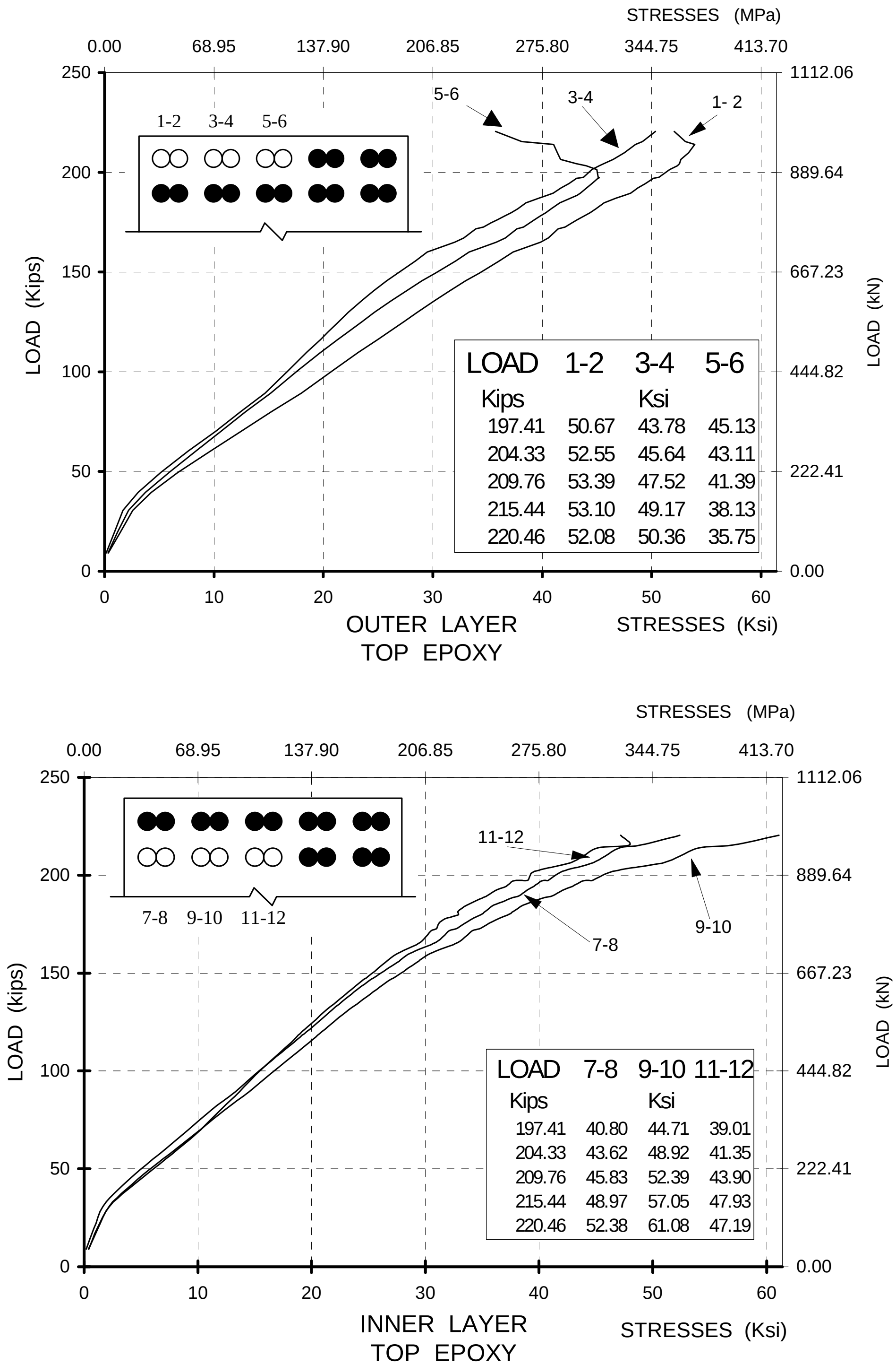

Figure 4.5.- Load versus Stresses of Bundled Bars Test: Top Epoxy

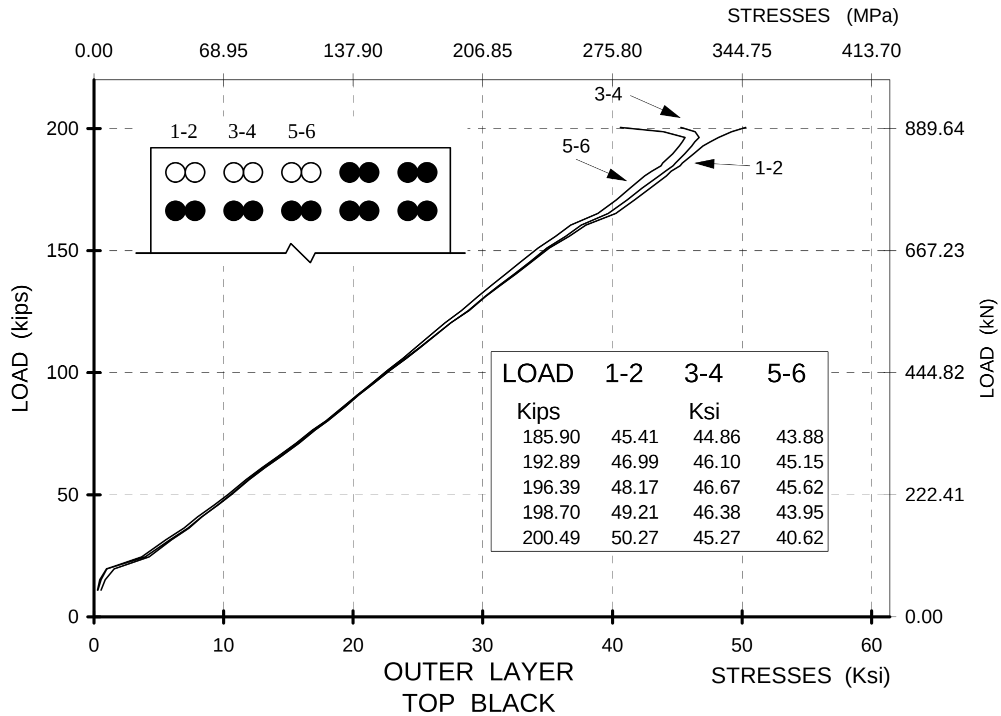

Figure 4.6.- Load versus Stresses of Bundled Bars
Test: Top Black

Figure 4.7.- Load versus Stresses of Bundled Bars
Test: Chen's Bottom

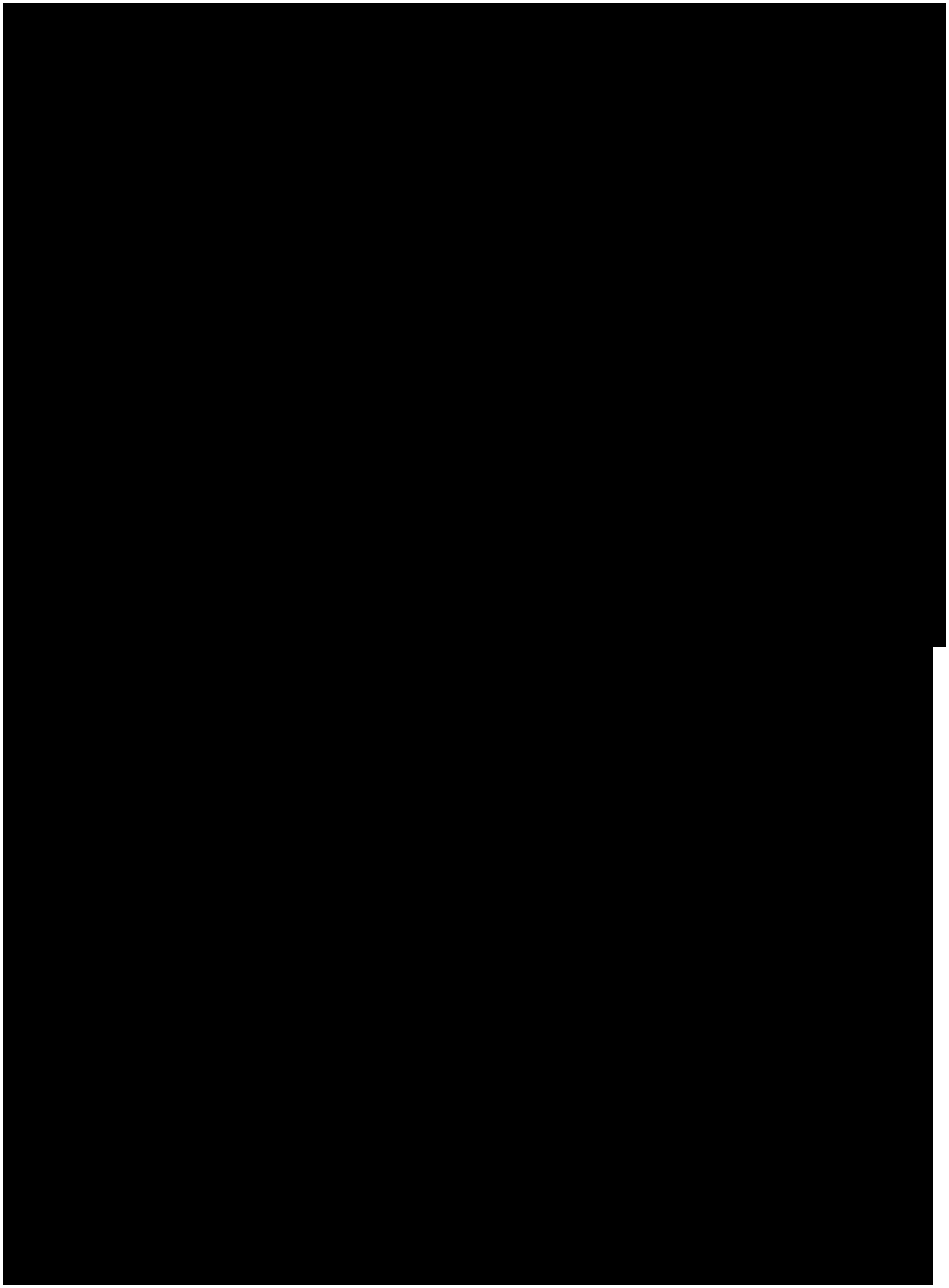

Figure 4.8.- Load versus Stresses of Bundled Bars.
Test: Bottom Epoxy

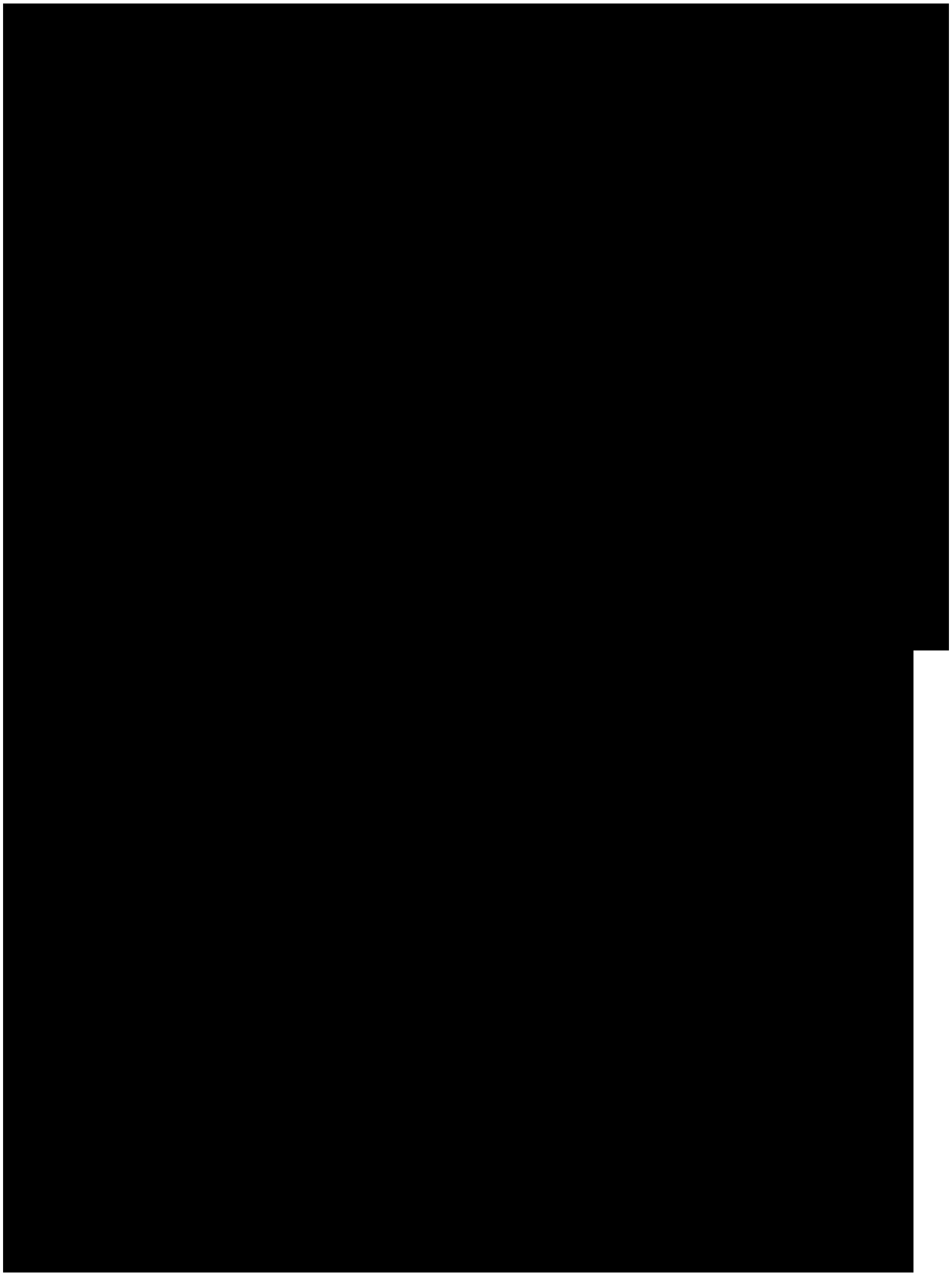

Figure 4.9.- Load versus Stresses of Bundled Bars
Test: Bottom Black

measured bond strength is 9.84, while for the top black test this strength is only 8.36. The bond strength increased about 18 %.

In the bottom casting position, no significant loss in bond was found when the bond strength averages were compared. However, bars in the inner layer of the uncoated test yielded. As shown in Table 4.3, the average bond strength for epoxy coated bars was 9.22 and 9.44 for the uncoated bars. The bond strength loss was only 2 %.

4.4.5.- Influence of Casting Position.

The ratio of bond strength for bottom cast versus top cast bars in the uncoated tests is 1.13. In Chen's test a ratio of 1.15 was obtained. The effect of casting position in the epoxy-coated bars was not determined since the top epoxy test produced higher results than normal.

4.5.- Development Lengths.

In order to make some comparison with current codes, development lengths based on the current AASHTO and ACI Code, as well as on the Orangun's formula are calculated. These lengths are compared with those obtained from tests by extrapolation. The conversion of the test results to development lengths is computed proportionally:

$$\frac{fs}{ls} = \frac{fy}{ld} \Rightarrow ld = \frac{fy}{fs} ls$$

where l_d is the development length, ls is the anchored length tested, and fs is the average stress measured in the bars.

Development lengths computed by the ACI Code include a 1.3 factor for top casting position and a 1.2 factor for epoxy coating. On the other hand, AASHTO specifications considered a 1.4 factor for top casting position and a factor of 1.15 for epoxy-coated bars.

The Orangun formula for the anchorage length was given in Equation 2.7. In this equation, the yielding stresses were used. The modified equation is as follow:

$$l_d = \frac{d_b \dfrac{f_y}{4\sqrt{f'c}} - 50}{1.2 + 3\dfrac{c}{d_b} + \dfrac{A_{tr} f_{yt}}{500 s d_b}}$$

Development lengths computed by this equation considered only a factor of 1.3 for top casting position.

As can be seen in Table 4.4, the ACI Code as well as the AASHTO specifications overestimates the development length for the top casting cases.However, the computed length from the top epoxy test is smaller than normal and therefore this overestimation is also smaller. In the top black case, the overestimation is 14 % for the AASHTO and 32 % for that recommended by the ACI.

For the bottom black test, the AASTHO underestimates the length by 10 % while the ACI overestimated it by 13 %. In the case of the bottom epoxy-coated bar

Table 4.4.- Development Length.

TEST	Development Length Ld in. (mm)			
	From Test	AASHTO	ACI	Orangun
TOP EPOXY	18.78 (477)*	29.66 (753)	35.83 (910)	21.44 (544)
TOP BLACK	22.62 (575)	25.79 (655)	29.85 (758)	22.07 (561)
BOTTOM EPOXY	20.05 (509)	21.18 (538)	27.56 (700)	16.49 (419)
BOTTOM BLACK	20.28 (515)	18.42 (468)	22.97 (583)	17.24 (438)

* Smaller length than normal.

test, the AASHTO resulted in a length 6 % greater than calculated from test results while the ACI length exceeded it by 37 %.

The length computed from the Orangun's formula resulted in 2.5% shorter length in the case of the top casting position and 15 % shorter for the bottom casting position in black tests. No comparisons were made with respect to the epoxy tests since no factor is considered in the formula.

Chapter 5

Conclusions

1.-Influence of Epoxy Coating.

The main purpose of this program was to study the influence of epoxy coating in bundled bars in several layers. One of the tests (top epoxy) resulted in bar stresses higher than expected, therefore no observations could be drawn with respect to the presence of coating in top casting bars. However, comparisons in the bottom casting position, with and without epoxy coating, showed that the measured bond strength averages were very close and no significant differences were found in comparison with black bars. The measured stresses in the outer layer of bars in both bottom tests were very close. At the peak load, higher stresses were developed in the inner layer of bars, and in the bottom black test, the stresses reached yielding. Yielding of these bars, allowed the section to carry higher loads.

2.- Casting Position.

Comparisons in the uncoated tests showed that the top casting bars developed 13 % less bond strength than that in the bottom casting position. This value was consistent with that obtained by Chen (15 % less).

3.-Development Lengths.

Development lengths were computed by extrapolation from stresses measured in the bars. These lengths were compared with those required by the AASHTO specification, by the ACI Code, and for those obtained from the Orangun's formula. For the case of uncoated bars in bottom casting position, the AASHTO specifications underestimate this length by 10 % while the ACI Code showed a value 13 % higher. For the case of epoxy-coated bars in bottom casting position the AASHTO specify a length slightly conservative while the ACI specified a length 37 % longer.

For the cases of uncoated bars, the Orangun formula gave a development length nearly the same for the top casting , however for the bottom casting position it resulted in a length 15 % shorter.

It should be mentioned that the basic development length in tension computed by the ACI Code equation was based on experimental evidence and that in order to ensure safety of the member this computed development length has been increased about 15%.

More tests would be helpful draw a better understanding of the behavior of bundles bars in multiple layers. However, there appears to be no reason to introduce special design provisions for epoxy-coated bars in bundles or in layers.

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