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BOND OF EPOXY COATED REINFORCING STEEL IN CONCRETE BRIDGE DECKS

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ABSTRACT

BOND OF EPOXY COATED REINFORCING STEEL IN CONCRETE BRIDGE DECKS

by

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The objective of this study was to evaluate the flexural bond characteristics of epoxy coated reinforcing steel in concrete bridge deck slabs under static loading. Behavior under service and ultimate loadings was considered.

Four series of two specimens each were tested with one specimen containing uncoated steel and one containing epoxy coated steel. Specimens were loaded in an inverted third-point loading with the reinforcing steel spliced at the midspan. Splice length was varied between tests.

Fewer and wider flexural cracks were found in specimens with epoxy coated steel relative to companion specimens with uncoated steel. A slight loss in stiffness also was seen in epoxy coated specimens. Epoxy coating caused a significant reduction in bond strength. This reduction, compared to companion uncoated bar specimens, increased with increasing anchorage length and increasing concrete strength.

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NOTATION

d = depth to longitudinal reinforcement, in.

d_b = diameter of reinforcing bar, in.

f'_c = concrete compressive strength, psi

f_s = stress in reinforcing bar, psi

f_y = yield stress of tension reinforcement, psi

f_{yt} = yield stress of transverse reinforcement, psi

l_{dp} = provided anchorage, in.

l_{db} = basic development length, in.

l_s = length of bar splice, in.

s = spacing of transverse reinforcement, in.

u = bond stress, psi

A_b = area of an individual bar, in.

A_{tr} = area of transverse reinforcement crossing plane of splitting, in.

C = lesser of C_b or C_s

C_b = thickness of concrete cover measured from extreme tension fiber to top of bar, in.

C_s = smaller of the concrete side cover or half the bar clear spacing, in.

I_{aux} = factor for auxiliary steel

K = Committee 408 factor for confinement

K_{tr} = Orangun, Jirsa, and Breen confinement factor

N = number of reinforcing bars

ϕ = strength reduction factor

CHAPTER 1: INTRODUCTION

1.1 Introduction

For the past several decades an area of major concern in detailing of concrete structures has been the adequate bond strength of longitudinal reinforcement. In the last fifteen years epoxy coated reinforcement has been used extensively in bridge decks as a positive means of preventing salt-induced corrosion. Recent studies have indicated decreased bond with such bars[7,8]. This reduction was found to be especially critical in splitting type bond failures. Splitting is especially significant when there is little concrete cover and no confinement from transverse steel. Some of these studies also found greater deflections and crack widths with the use of epoxy coated bars[7]. These findings have led to proposed revisions to the current design specifications for development of reinforcement in the American Concrete Institute Building Code[1]. If approved, such revisions will require, under special circumstances, longer development lengths for epoxy coated bars.

In snow-belt regions of the United States epoxy coated reinforcement is usually specified for use in bridge decks. Bridge deck members typically have concrete covers of two to three inches and little transverse confining steel. This situation has the potential for a splitting type bond failure. The only study of slab members with epoxy coated bars[7] was conducted with very large development lengths which resulted in flexural failures rather than splitting bond failures thus preventing evaluation of actual bond strength.

This study is an evaluation of the performance of contact lap splices of epoxy coated reinforcing steel in slab specimens representative of bridge deck members subjected to splitting type failures. Included are evaluations of the ultimate strength of the members as well as stiffness and cracking under service load conditions. The purpose of this study is to compare the bond of epoxy coated reinforcing bars with that of uncoated bars in bridge deck type members.

1.2 Use of Epoxy Coated Reinforcement

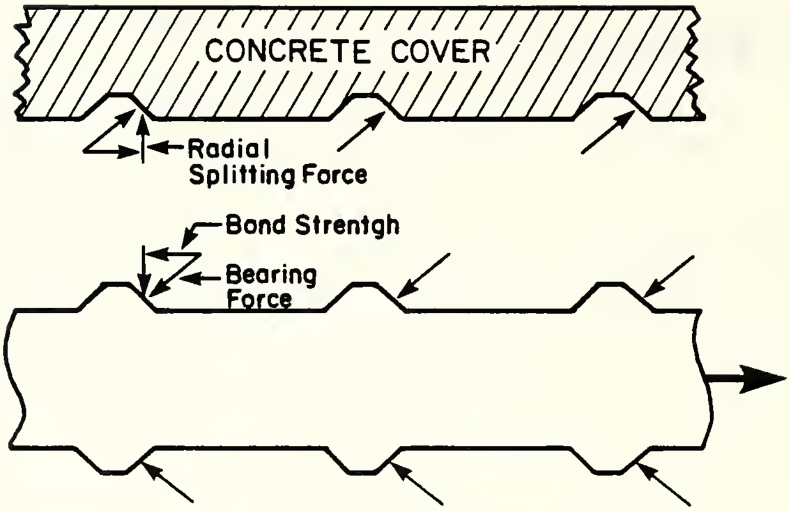
Epoxy coated reinforcing steel is used to prevent premature deterioration of concrete structures caused by chloride induced corrosion of the steel. Chloride ions reach the reinforcing steel through cracks in the concrete. The steel corrosion products occupy up to twenty times the volume of the original steel lost. This expansion produces a radial pressure on the concrete which causes cracking and spalling. De-icing salts and seawater spray are sources of chloride ions.

Originally, epoxy coated bars were primarily used in bridge decks exposed to de-icing salts. In addition to bridge decks, they are now employed in all types of concrete structures in corrosive environments. Applications include coastal structures, sewage treatment plants, and chemical plants.

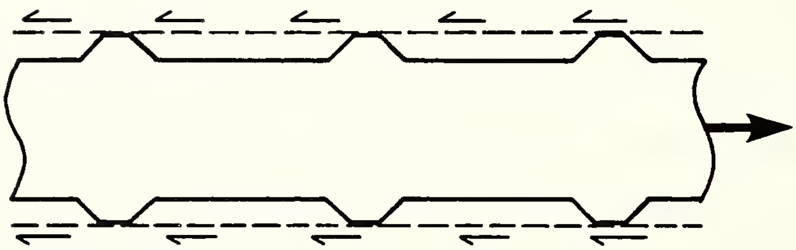
1.3 Review of Bond

In the design and analysis of reinforced concrete structures strain compatibility between the concrete and reinforcing steel is assumed. This assumption implies adequate bond between the steel and concrete. The following sections present a brief review of the current state of knowledge of bond in reinforced concrete.

1.3.1 **Modes of Bond Failure** There are two generally recognized modes of bond failure - splitting failure and pullout failure. Components of the bond mechanism include adhesion of the concrete to the steel, friction between the concrete and steel, and bearing of the bar deformations against the concrete. This bearing, acting normal to the face of the deformations, is the primary component of bond. The component of the bearing reaction perpendicular to the axis of the bar exerts a radial pressure on the surrounding concrete as shown in Figure 1.1a. If bar spacing or cover are relatively small, the tensile strength of the concrete may be exceeded and a splitting failure can occur. This failure can be in the form of a side split failure, a face-and-side split failure, or a V-notch failure (Figure 1.2). Restraint against splitting is mainly dependent on concrete tensile strength and the amount of transverse reinforcement crossing the splitting plane. The size of bar, spacing and concrete cover also influence splitting as discussed in



a) splitting



b) pullout

Figure 1.1 Stresses in Bond Failures a) splitting, b) pullout (Treece and Jirsa).

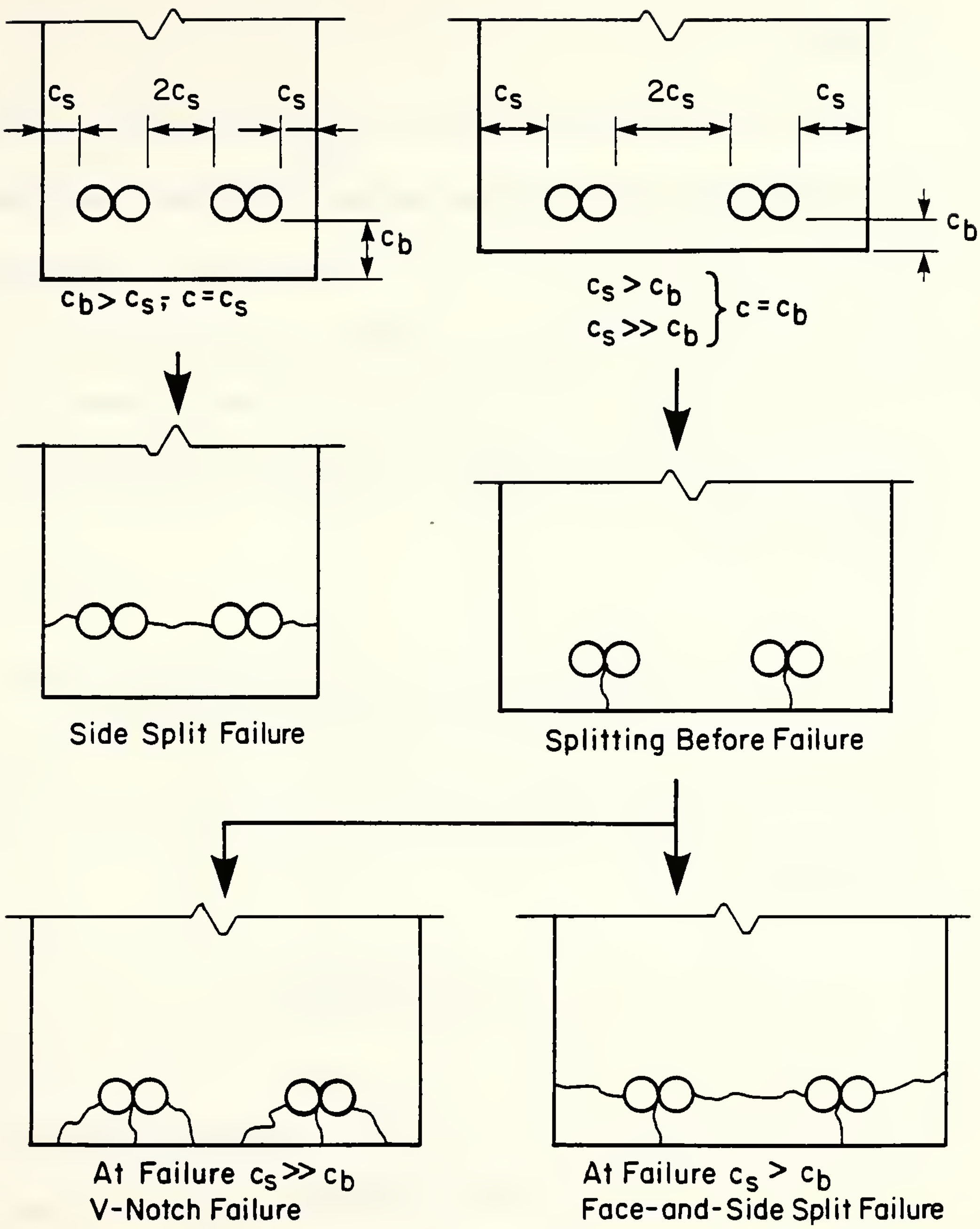


Figure 1.2 Splitting Bond Failures (Orangun et al).

Section 1.3.3.

When cover or one-half the clear spacing between bars is approximately three times greater than the bar diameter, radial bar forces do not totally split the surrounding concrete. Rather the component of the bearing reaction parallel to the bar axis causes a shearing from the surrounding concrete of the concrete between the bar deformations as shown in Figure 1.1b. This type of failure is called a pullout failure and is primarily dependent on the concrete shearing strength.

1.3.2 **ACI Provisions** Current ACI 318 design provisions[1] require a basic development length to ensure yield of the steel at the critical section. This is to provide member ductility. For #11 bars and smaller the requirement is

$$l_{db} = 0.04 A_b \frac{f_y}{\sqrt{f'_c}} \tag{1}$$

and

$$l_{db} \geq 0.0004 d_b f_y . \tag{2}$$

The basic development length is modified by factors to account for top reinforcement, yield strength of the steel, lightweight concrete, bar spacing and excess reinforcement. These provisions were developed assuming that the bar stress must reach 1.25 times its yield strength to account for reduction in bond stress due to splitting action.

Equations (1) and (2) are derived by equating the bond strength over the surface of the bar, $u\pi l_{db} d_b$, to the force in the bar, $1.25A_b f_y$, and setting the bond stress, u, to $9.5\frac{\sqrt{f'_c}}{d_b}$ for a splitting failure or to 800 psi for a pullout failure[2,4].

For splices in tension, ACI 318-83 Section 12.15 provides for lap splice lengths in terms of l_d. The development length is modified by a factor of 1.0 to 1.7 depending on the percentage of steel to be spliced and the stress to be developed.

1.3.3 **Other Studies in the United States** Although concrete cover, and the amount of transverse reinforcement are known to influence bond strength, these parameters are not currently reflected in the ACI Code basic development length provisions.

In 1977, Orangun, Jirsa, and Breen proposed an equation for the bond stress of deformed bars based on concrete cover, bar spacing[2] and amount of transverse reinforcement. The equation was developed through a non-linear regression analysis of data from previous splice tests.

In the equation

$$\frac{u}{\sqrt{f'_c}} = 1.2 + \frac{3C}{d_b} + \frac{50d_b}{l_s} + K_{tr} \qquad (3)$$

C is taken as the smaller value of the bottom cover or one-half the clear spacing. When the bottom cover is greater than one-half the clear spacing, a side split failure is expected. When the bottom cover is less than half the clear spacing, a face-and-side split or a V-notch failure is expected (see Figure 1.2). K_{tr} is a factor reflecting the confinement provided by transverse reinforcement. It was found to be

$$K_{tr} = A_{tr} \frac{f_{yt}}{500 s d_b}. \tag{4}$$

Based on the work of Orangun, Jirsa, and Breen, ACI Committee 408 published a new recommendation for development lengths[3,4]. In a departure from previous ACI 318 provisions, splice length and development length were the same in the Committee 408 recommendation. The validity of this has been confirmed by Reynolds and Beeby[11]. This recommendation provides a rational basis for computation of development lengths. However, it has not been adopted by Committee 318.

The Committee 408 recommendations are as follows:

for clear cover greater than 2" and center-to-center bar spacing greater than 5";

$$l_{db} = \frac{23 d_b}{\phi} \qquad \text{(\#6 bars and smaller)} \tag{5}$$

$$l_{db} = \frac{2200 A_b}{\phi \sqrt{f'_c}}. \qquad \text{(\#7 bars and larger)} \tag{6}$$

For other situations or to consider the influence of transverse steel;

$$l_{db} = \frac{5500A_b}{\phi K\sqrt{f'_c}} \qquad \text{(other cases)} \tag{7}$$

where K equals the smaller of C_c+K_{tr} or C_s+K_{tr} and $K_{tr} = A_{tr}f_{yt}/1500s$. The factor ϕ is to be taken as 0.8.

In 1985 Kemp[13] recommended the following equation for bond strength

$$u = 232 + 2.716\frac{C}{d_b}\sqrt{f'_c} + .201A_{tr}\frac{f_{yt}}{sd_b} + 195.0I_{aux} + 21.16(F_dN)^{.66} \tag{8}$$

The equation was based on the results of 157 cantilever beam stub tests in which a longitudinal and a dowel force is applied directly to the reinforcing steel as shown in Figure 1.3. In this equation I_{aux} is either 1 or 0 depending whether auxiliary steel is present. Auxiliary steel was provided in some of Kemp's test specimens to resist the large shearing forces and bending moments produced in relatively small beams. Auxiliary steel is not normally present in practice. F_d is the dowel force in each bar. Dowel force is the force of the bar bearing against the concrete perpendicular to the longitudinal axis of the steel. This force is due to bending in the beam. It occurs when a shear crack develops in the concrete. Dowel forces were found to have little influence on bond stress until they are close to the ultimate dowel capacity of the member.

If $\sqrt{f'_c}$ is factored from each term of the expression and the terms for the auxiliary steel and dowel force are neglected, the expression is essentially the same as that of Orangun, Jirsa, and Breen. The primary difference being Kemp does not consider the influence of the provided anchorage length.

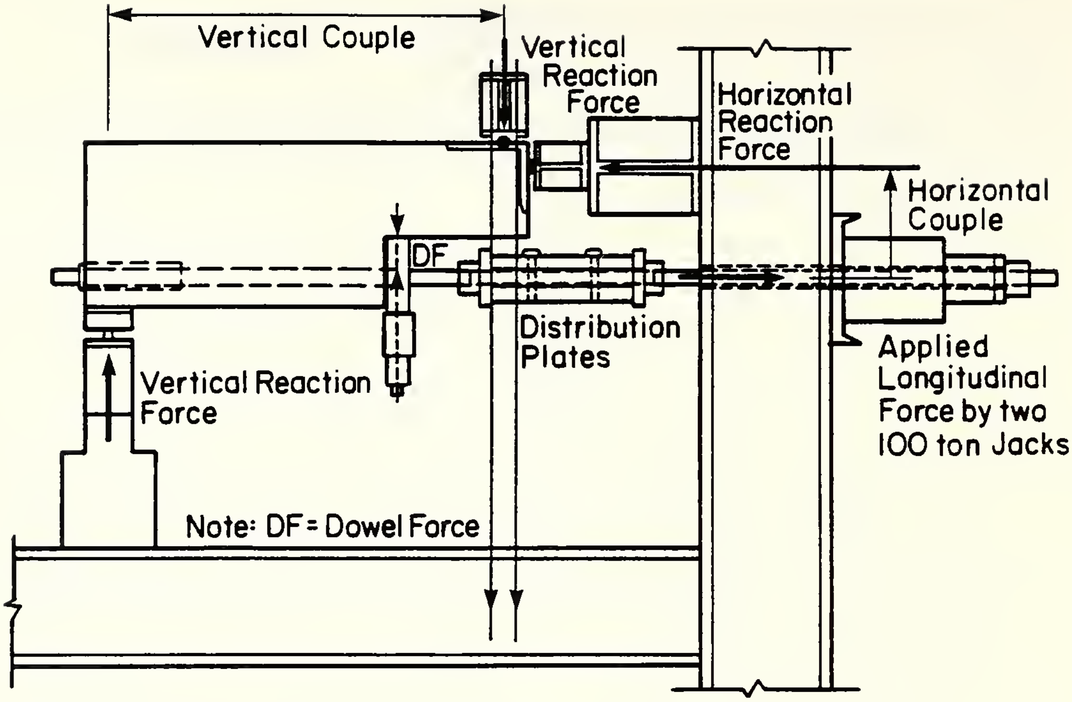


Figure 1.3 Modified Cantilever Beam Stub (Kemp).

The existing and proposed design specifications were developed on the basis of test results with uncoated reinforcement. With the widespread introduction of epoxy coated reinforcing, because of the empirical nature of such design specifications, their applicability needs to be investigated. In fact ACI 318-83 specification 7.4.1 states that "...reinforcement shall be free from mud, oil, or other non-metallic coatings that adversely affect bonding capacity[1]."

1.3.4 Potential Influence of Epoxy Coating on Bond Strength

Epoxy coatings may not produce bond strength reductions in pullout type failures. Although some radial splitting due to bearing may occur. the primary mode of failure is shearing of the concrete between the ribs from the surrounding concrete. This shearing failure force would be nearly the same in epoxy coated or uncoated bars.

In a splitting failure, stresses produced by bearing of the ribs on the concrete exceed the tensile capacity of the surrounding concrete. Previous work[7,8] has shown little adhesion between epoxy coated reinforcing steel and concrete. This will result in increased bar slip producing a wedging action which drives the concrete apart. This trend could lead to reduced bond strengths with epoxy coated bars when concrete splitting is the mode of failure.

1.4 Previous Research

1.4.1 National Bureau of Standards Studies

In 1976 Mathey and Clifton of the National Bureau of Standards (NBS) reported epoxy coated bars with a 1 mil to 11 mil coating thickness had a 6% lower critical bond strength than uncoated bars[5]. This slight reduction was deemed acceptable.

The NBS tests were pullout tests on 10x10x12 in. concrete prisms. #6 bars were embedded for the entire 12 inches. Critical bond stress was defined as that stress producing a free-end slip of 0.002 in. or a loaded-end slip of 0.01 in. These values were based on previous work[6] showing that in general significant changes in the slope of the bond stress-slip relationship occurred at these values of slip for various lengths of embedment in beams with #4 and

#8 bars.

Cracking perpendicular to the reinforcing steel will not occur in a pullout test. In pullout tests, after development of the steel, the bar stress is constant. However, in a beam the bar stress is greatest at crack locations[12]. The pullout tests do not represent the stress concentration occurring at crack locations in beams. A pullout test such as used in the NBS study will not indicate a reduction in bond strength which could conceivably be caused by an increase in wedging action due to a possible reduction in adhesion with epoxy coated reinforcement.

In addition, the prisms used in the study included a welded wire fabric cage to control splitting. Additional confinement at the end regions of the specimens was provided by friction against the loading plate. These facts make such pullout specimens of questionable value in evaluating bond strength other than for pullout failures.

1.4.2 **North Carolina State Studies** In 1982 Johnston and Zia of North Carolina State University conducted a series of tests on slab specimens and beam stub specimens to evaluate the bond characteristics of epoxy coated reinforcing bars[7]. Based on these tests a 15% increase in the development length was recommended when epoxy coated bars are used.

The slab specimens were designed primarily for crack width and spacing comparisons. The test specimens were 6 ft. long, 2 ft. wide and 8-1/2 in. deep.

Reinforcement consisted of two layers of 3-#6 bars at 8 in. with 2-7/8 in. top cover and 3-5/8 in. side cover on the top layer of steel. The slab specimen details are shown in Figure 1.4. They were tested as simple beams with 4 ft. between support points as shown in Figure 1.5. This arrangement provided a development length of 35 in. Specimens using epoxy bars were compared to those using uncoated bars. Slabs with epoxy coated bars were found to have greater crack widths and deflections than those with uncoated steel. There was a 4% increase in strength for mill scale specimens compared to epoxy coated specimens.

The simple support set-up used in the North Carolina State studies produces a steep moment gradient which does not allow for the random formation of cracks. This may have influenced the crack width and stiffness comparisons. Also, the 35 in. development length was over two times that required by current code specifications. This resulted in flexural failures making the evaluation of bond strength with these specimens difficult.

The beam stub specimens (Figure 1.6) attempt to simulate beam behavior although the load is applied directly to the reinforcing steel. These beams contained either a #6 or #11 bar with a cover to bar diameter ratio ranging from 1.46 to 3.10. Transverse reinforcement was provided by #3 stirrups spaced at 3 in. or 6 in. as shown in Figure 1.7. These tests were terminated at 1.25 to 1.40 times the yield stress of the bars. In the specimens in which splitting failures occurred the epoxy coated bars developed 85% of the bond of

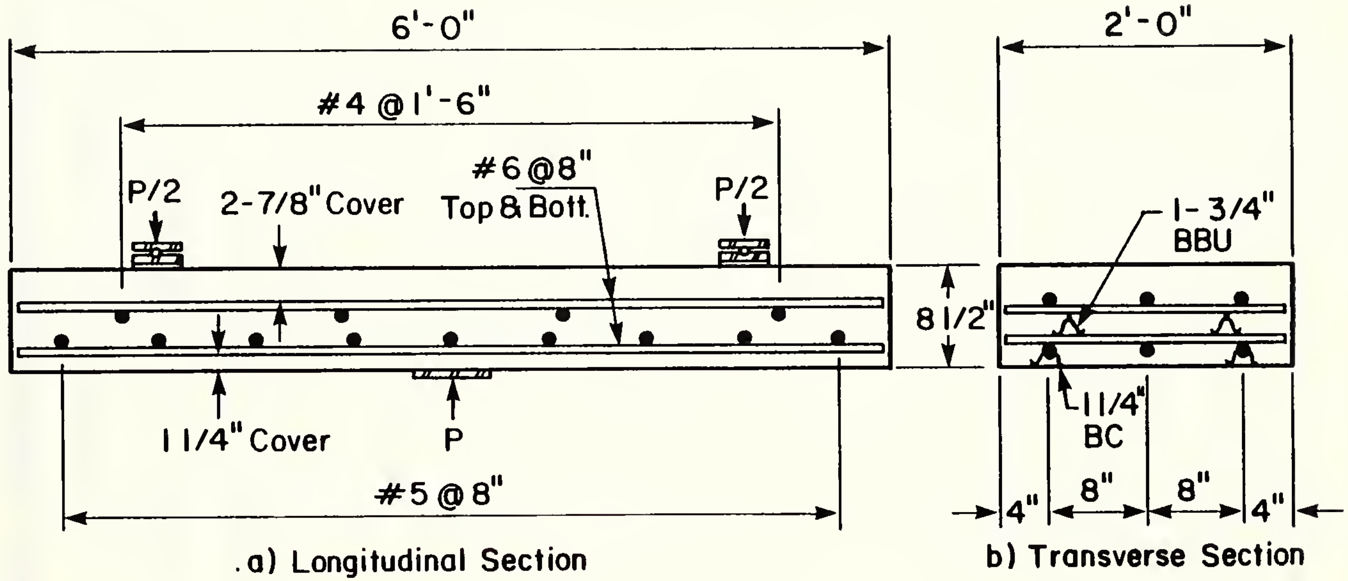


Figure 1.4 North Carolina State Slab Specimen Details (Johnston and Zia).

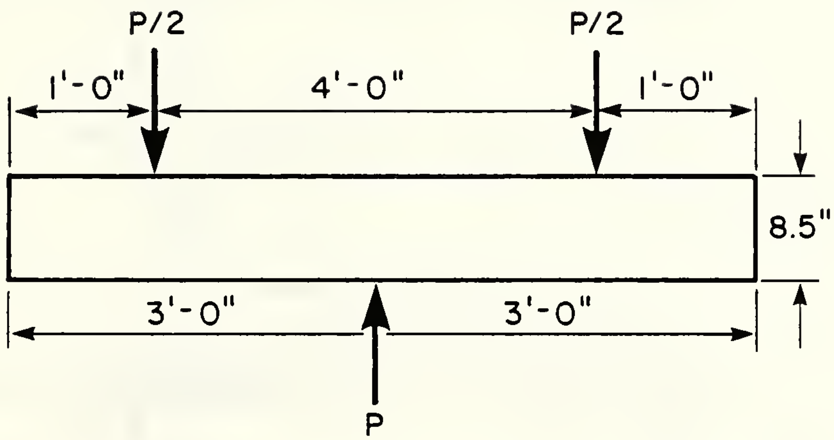


Figure 1.5 North Carolina State Slab Loading.

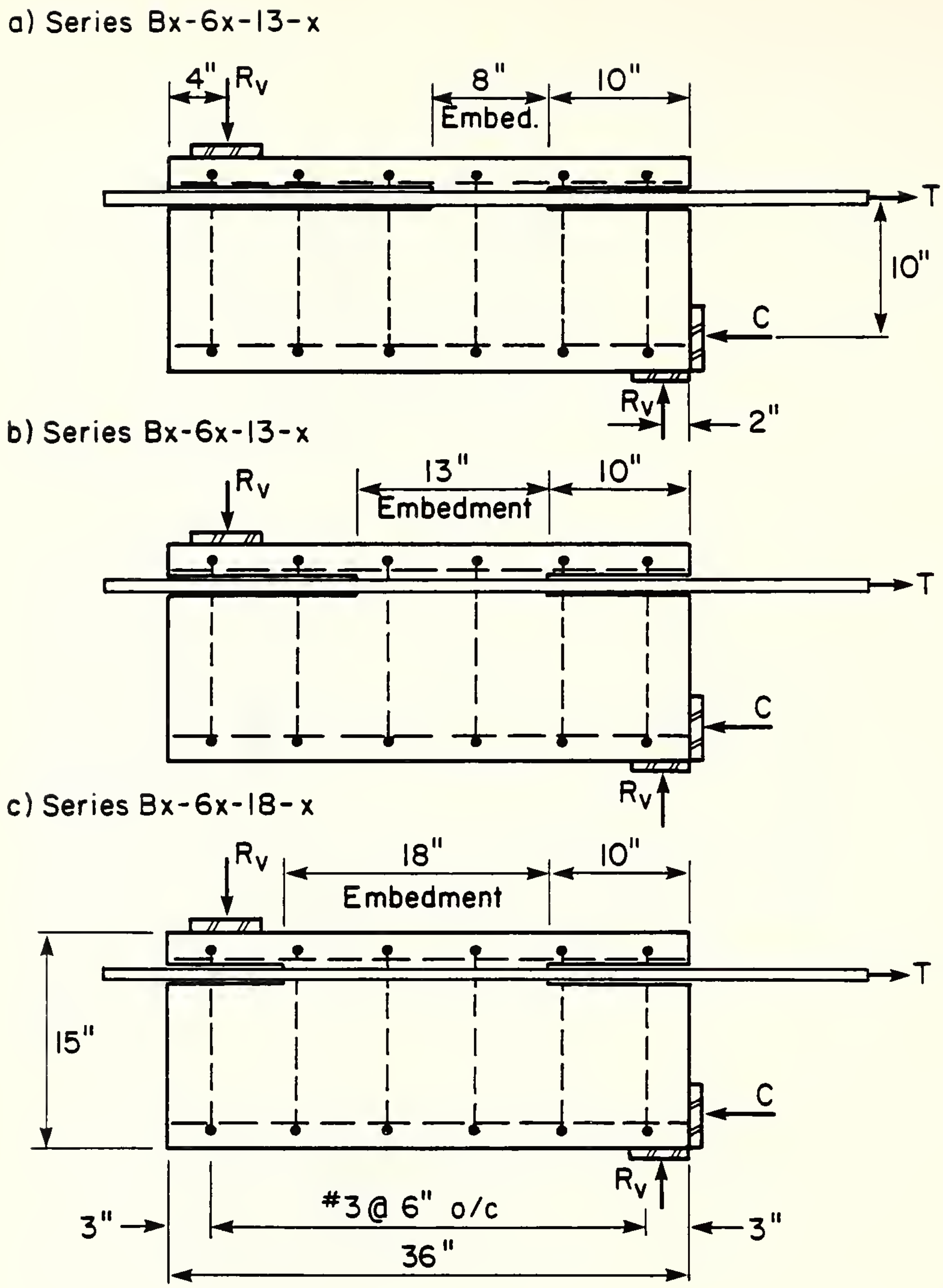


Figure 1.6 North Carolina State Beam Stub Specimens (Johnston and Zia).

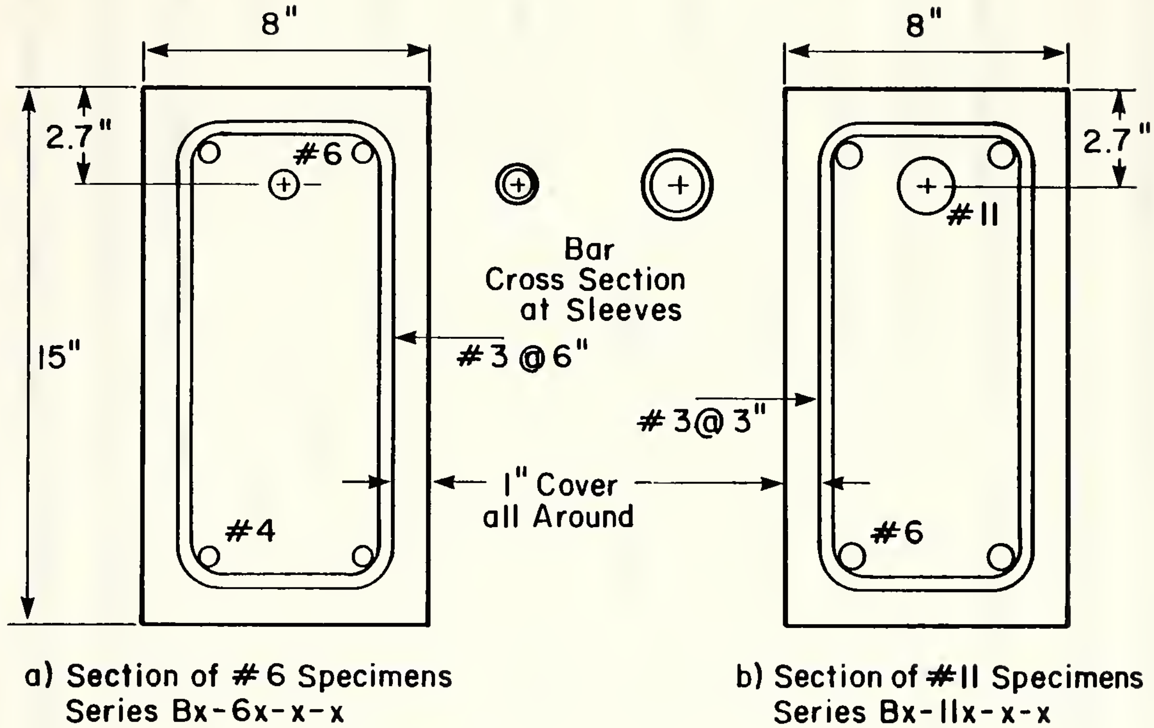


Figure 1.7 North Carolina State Beam Stub Cross Sections (Johnston and Zia).

the uncoated bars. The epoxy coated bars did develop 1.25 times the yield strength of the steel as intended by Code specifications. A 15% increase in development length was recommended for epoxy bars to provide behavior comparable to that of uncoated bars.

These beam stub specimens contained transverse reinforcement in the form of U-stirrups which cross the plane of splitting between the longitudinal bars and the concrete surface. A typical bridge deck will contain transverse steel; however, it will generally be at a wider spacing. As noted in Section 1.3.3, transverse steel can have a significant influence on bond strength. The transverse steel may have a stapling effect.

The loading system used produced a linear moment diagram. This again prevents the random formation of cracks as occurred with the slab tests. The influence of multiple bars and bar spacing was also not considered as all the specimens only contained a single bar.

1.4.3 **University of Texas at Austin Studies** In 1986 Treece and Jirsa of the University of Texas at Austin also studied the bond of epoxy coated reinforcing steel[8]. The influence of bar size, concrete strength, casting position, and epoxy coating thickness on bond was considered. In these tests it was found that epoxy coating was the only variable which caused a reduction in bond strength. Epoxy coated reinforcement developed approximately 66% of the bond developed in uncoated steel. Crack widths were 50% greater in the coated bar specimens than in those with uncoated bars although there were fewer cracks. There was not a corresponding loss of stiffness.

Based on their work and that of Johnston and Zia[7], Treece and Jirsa recommended a 50% increase in basic development length in situations where the concrete cover is less than $3d_b$ or bar spacing is less than $6d_b$. For all other situations they recommended a 15% increase in basic development length. In bridge decks the concrete cover is usually 2 to 3 bar diameters thick and there is a low amount of transverse steel crossing the splitting plane. Therefore the 50% reduction would apply. These recommendations will be adopted by the ACI in the 1989 Building Code[14] with the exception that a 20% increase in development length will be recommended over the 15% recommended by Treece.

The factor of 1.5 proposed by Treece and Jirsa is the reciprocal of the 66% bond strength found for epoxy coated bars relative to the strength of uncoated bars. The actual test values ranged from 87% to 54%. In the study no influence on bond reduction due to concrete strength or provided anchorage length was found. This may be because in reducing their data, differences in these two parameters between tests were not properly normalized. In the studies by Treece and Jirsa, normalization was conducted by taking the ratio of the bond stress in each test to the value of bond stress predicted by the equation developed by Orangun, Jirsa, and Breen (Equation 3). These two ratios were then compared. By initially taking the ratio of test bond stress to the empirical bond stress it was felt that differences in cover to bar diameter, splice length, and concrete strength between test specimens were normalized. However, Equation (3) was developed through nonlinear regression analysis of test data from uncoated bars only. The weighted influence of the individual test parameters suggested by such an equation only have meaning for uncoated reinforcing bars. The relative influence of each parameter on the bond stress developed by an epoxy coated bar may be different. Therefore the normalized values of bond stress produced by this procedure are questionable.

The normalization procedure used by Treece and Jirsa did not effect the 66% average bond ratio value because the most significant variation of test parameters occurred in series in which the steel yielded. Consequently those specimens were not included in the average bond ratio value. It is important to note, however, that the limits for application of the 50% development length increase were established through tests on uncoated reinforcing bars. It is conceivable that splitting failures could occur with epoxy coated reinforcing bars at cover values higher than $3d_b$.

The University of Texas at Austin tests were performed on two sizes of beams containing either three #6 or three #11 bars. Details of these beams are shown in Figure 1.8. They were loaded in an inverted two-point bend as shown in Figure 1.9. Reinforcement was spliced in the constant moment region. The splices were designed to fail prior to yielding of the steel in order

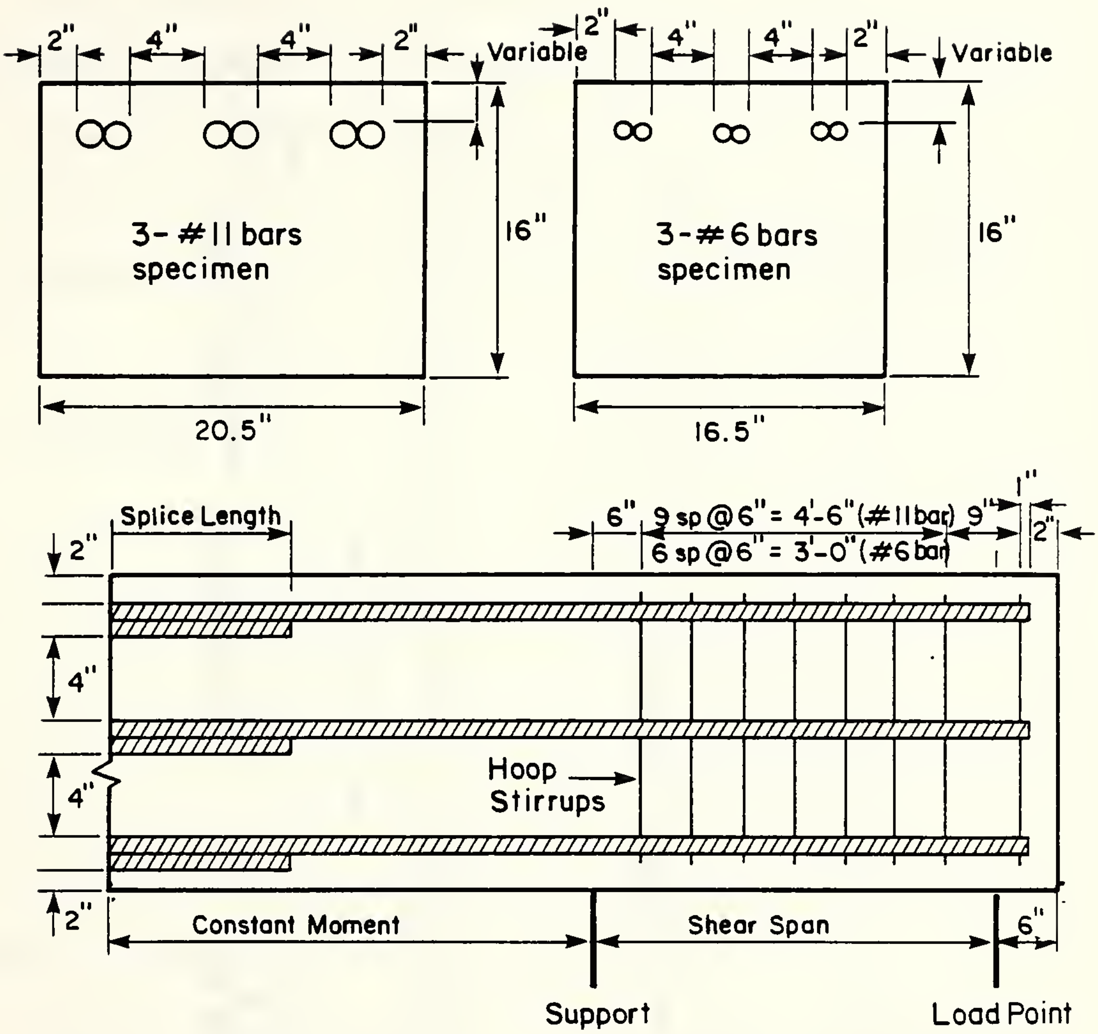


Figure 1.8 University of Texas at Austin Beam Details (Treece and Jirsa).

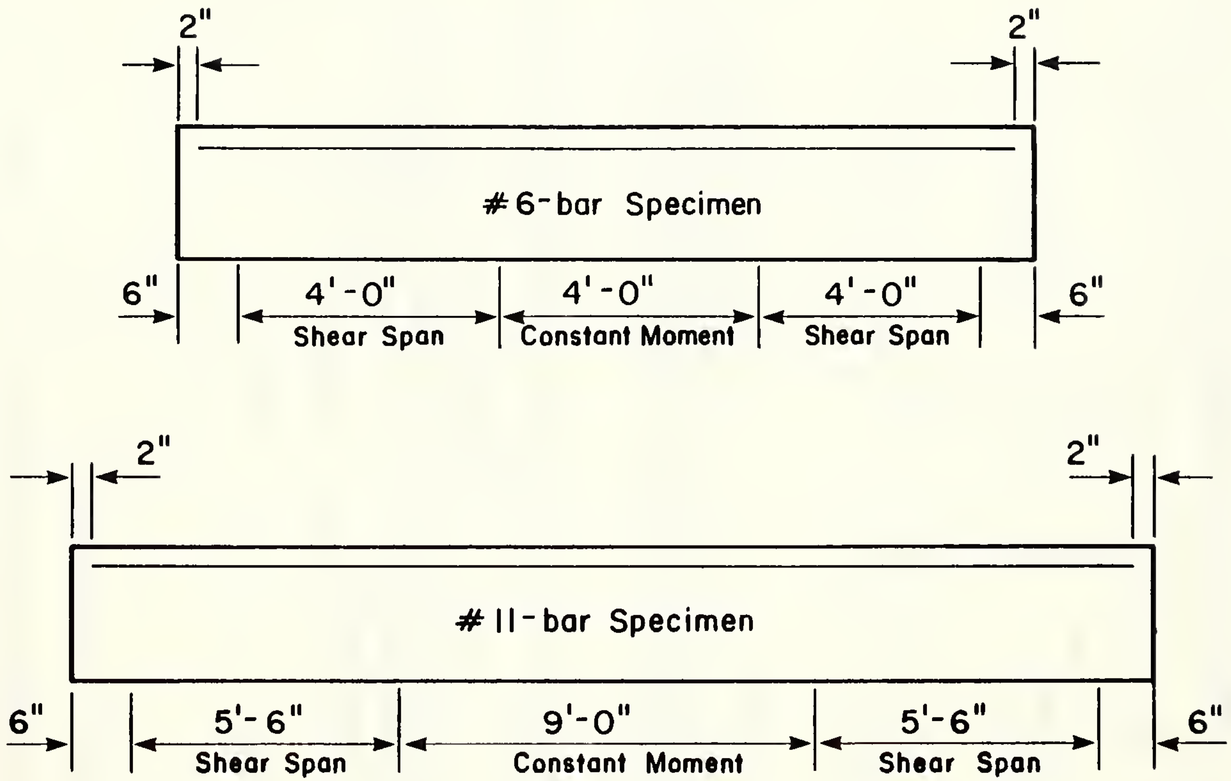


Figure 1.9 University of Texas at Austin Beam Loading (Treece and Jirsa).

to evaluate the bond strength.

These beams did not contain transverse steel crossing the failure plane. However, in order to produce splitting failures prior to yield the cover used was approximately 3/4 in. This cover is much lower than that used in bridge deck slabs. Top casting may also have an effect. In the Texas studies top casting could not be evaluated because of the use of low slump concrete.

1.5 Summary

Concern toward adequate detailing in concrete structures has led researchers to a better understanding of and provisions for development of reinforcing steel. Orangun, Jirsa and Breen[2] have provided an empirical equation for bond stress and thus development length for deformed reinforcing steel. Because of the highly empirical nature of such design recommendations their extrapolation to different types of reinforcing steel, such as epoxy coated bars, may be unjustified.

The more recent introduction of epoxy coated reinforcing steel for corrosion control has led to investigations of its bond characteristics as well as the behavior of members which use such reinforcement. Studies[7,8] have shown a reduction in bond strength with epoxy coated reinforcement. This reduction is especially significant in splitting type bond failures.

Recommendations have been made, based on these studies, to modify current design codes. These recommended modifications will effect all types of members including bridge deck slabs. Although epoxy coated reinforcing steel is used extensively in bridge decks, no bond studies have been conducted on splitting failures in this type of member with epoxy coated steel. The beam studies conducted with epoxy bars do not truly evaluate their use in slabs. The previous beam tests contained either transverse reinforcement crossing the splitting plane or very small concrete covers. Both of these factors have been shown to significantly affect bond.

Chapter 2 is a presentation of the experimental program used to evaluate the bond strength of epoxy coated bars in bridge decks. The experiments considered the influence of random crack formation, anchorage length, and concrete strength on members which have concrete cover similar to that found in bridge decks.

CHAPTER 2: EXPERIMENTAL PROGRAM

2.1 Introduction

As the use of epoxy coated reinforcement has increased it has become necessary to evaluate the behavior of members using epoxy coated reinforcement and the adequacy of current design provisions relating to such reinforcement. Bridge deck members, with cover to bar diameter ratios typically less than three and with little or no transverse reinforcing steel crossing the splitting plane, have the potential for splitting type bond failures if detailing is inadequate. Studies by Treece and Jirsa[8] have shown a significant reduction in bond strength with the use of epoxy coated reinforcement in members subject to splitting type bond failures. This has led to suggested changes in Code recommendations for epoxy coated bar development. Because these changes would apply to bridge deck members as well, it is essential to determine if they are justified. The following chapter describes an experimental program to evaluate the performance of bridge deck members reinforced with epoxy coated bars both at ultimate and under service loads.

2.2 Purpose and Scope of Experimental Program

The experimental program was designed to evaluate the bond strength of splices of epoxy coated reinforcement in constant moment regions of slab specimens representative of bridge deck slabs. The evaluation includes ultimate strength comparisons as well as comparisons of crack width and deflection throughout the entire range of test loadings.

For this purpose four series of specimens were tested. Each series consisted of a 13ft x 2ft x 8in. slab reinforced with epoxy coated bars and a companion slab with uncoated reinforcement. The reinforcement was spliced at midspan, with the splice length being the main variable. The specimens were subjected to static loading at the ends and supported at the third points.

2.3 Design of Specimens

2.3.1 **Dimensions** The specimens tested were 13ft x 2ft x 8in. The design is shown in Figure 2.1. From discussions with highway bridge designers in Indiana, #6 bars at 8 in. was determined to be typical of bridge deck slab reinforcement. No transverse reinforcement was included. In order to place three bars a 2 ft slab width was selected. It was desired to include three bars to note any differences in steel stress between the center and edges of the member and to study the effect of bar spacing as well as cover. As the floor anchors of the laboratory are spaced on a 6 ft. grid, a 13 ft. member length

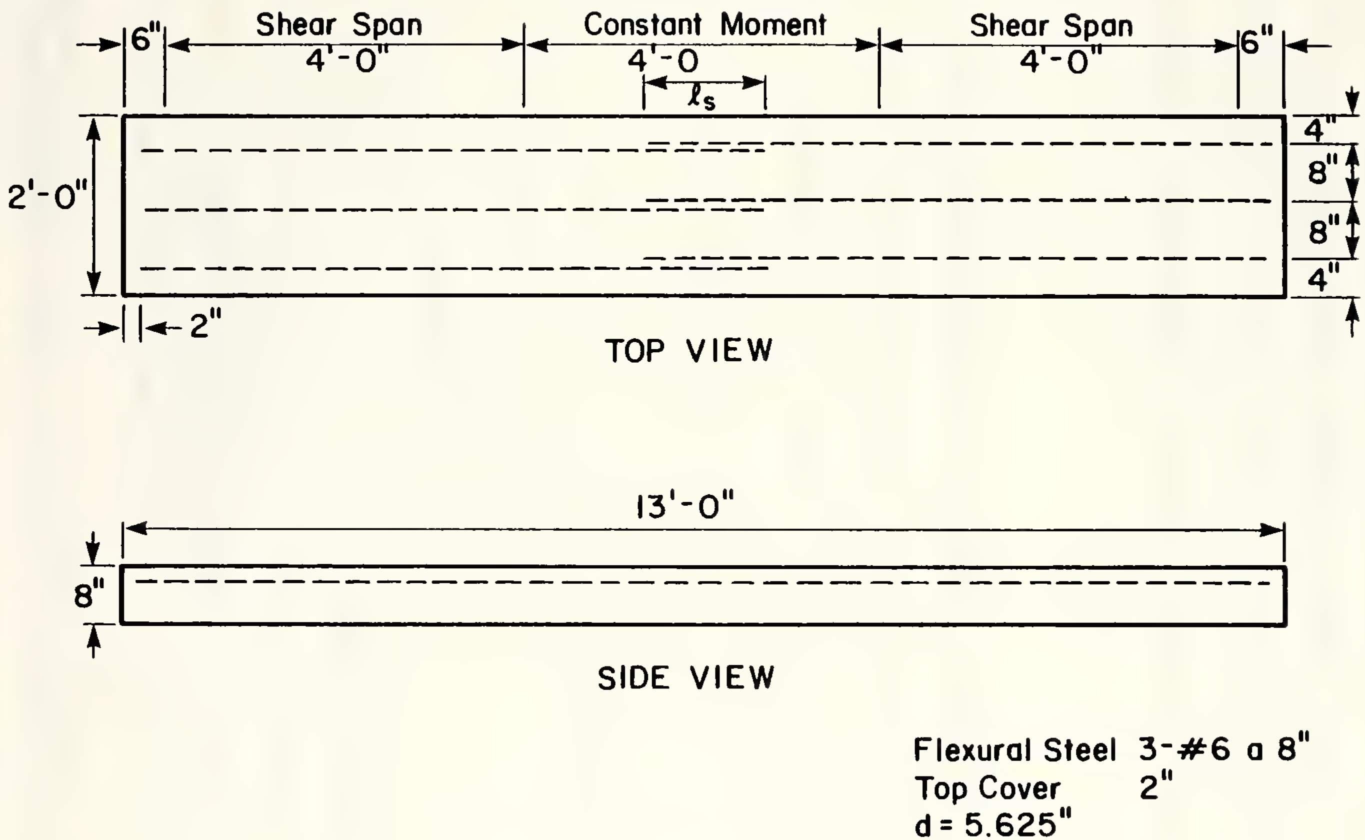


Figure 2.1 Slab Specimen Details.

was selected allowing 6 in. at each end for loading plates.

In order to study splitting failures it was necessary to minimize the concrete cover. American Association of State Highway and Traffic Officials (AASHTO) standards allow a minimum of 2 in. cover for bridge decks[9]. This was selected as the cover. This cover produces a cover to bar diameter ratio of 2.67.

2.3.2 **Design of Splices** Bond strength comparisons can be best accomplished if the reinforcing steel does not yield. Splice lengths were designed using the equation of Orangun, Jirsa, and Breen[2]

$$\frac{u}{\sqrt{f'_c}} = 1.2 + \frac{3C}{d_b} + \frac{50d_b}{l_s} + K_{tr} \tag{1}$$

with the bond stress, u, equal to

$$u = f_s \frac{d_b}{4l_s} \tag{2}$$

and solving for l_s giving

$$l_s = \frac{(f_s - 200\sqrt{f'_c})d_b}{4(1.2 + \frac{3C}{d_b} + K_{tr})\sqrt{f'_c}}. \tag{3}$$

The longest splice was selected by setting f_s to 60,000 psi and f'_c to 4000 psi giving a splice length of 15.26". A 16" inch length was used. A 14" and 12" splice were next tested using nominal 4000 psi concrete. A final series with a

10" splice and 8000 psi concrete was also tested in order to evaluate the influence of high strength concretes on bond of epoxy coated bars.

The splices tested were contact lap splices tied in the center of the specimen with two plastic ties. Care was taken to ensure the deformations were aligned in the same way for each splice and in each series. The bars were placed with the longitudinal rib up and the deformations parallel.

2.3.3 **Loading Arrangement** The slab specimens were loaded in an inverted third-point loading (Figure 2.2). The load was applied 6 in. from the ends with 12,000 pound capacity hydraulic rams mounted to a test frame. The members were supported 4 ft. from the loading points. The arrangement placed the splices in a constant moment region allowing for the random formation of cracks. This arrangement also provided for convenient observation and measurement of developing cracks.

The load was applied through 12x6x1/4 in. plates six inches from the ends of the member. The members were supported on concrete blocks and a 3/4 in. thick grooved plate with 2 in. rollers. A 24x3 in. hardboard plate was used to distribute the member reaction over the roller. In the 16 series 24x4x1/4 in. plates were used in place of the grooved plates. Floor tiedowns carried the frame reaction. Figure 2.3 shows a slab in place for testing.

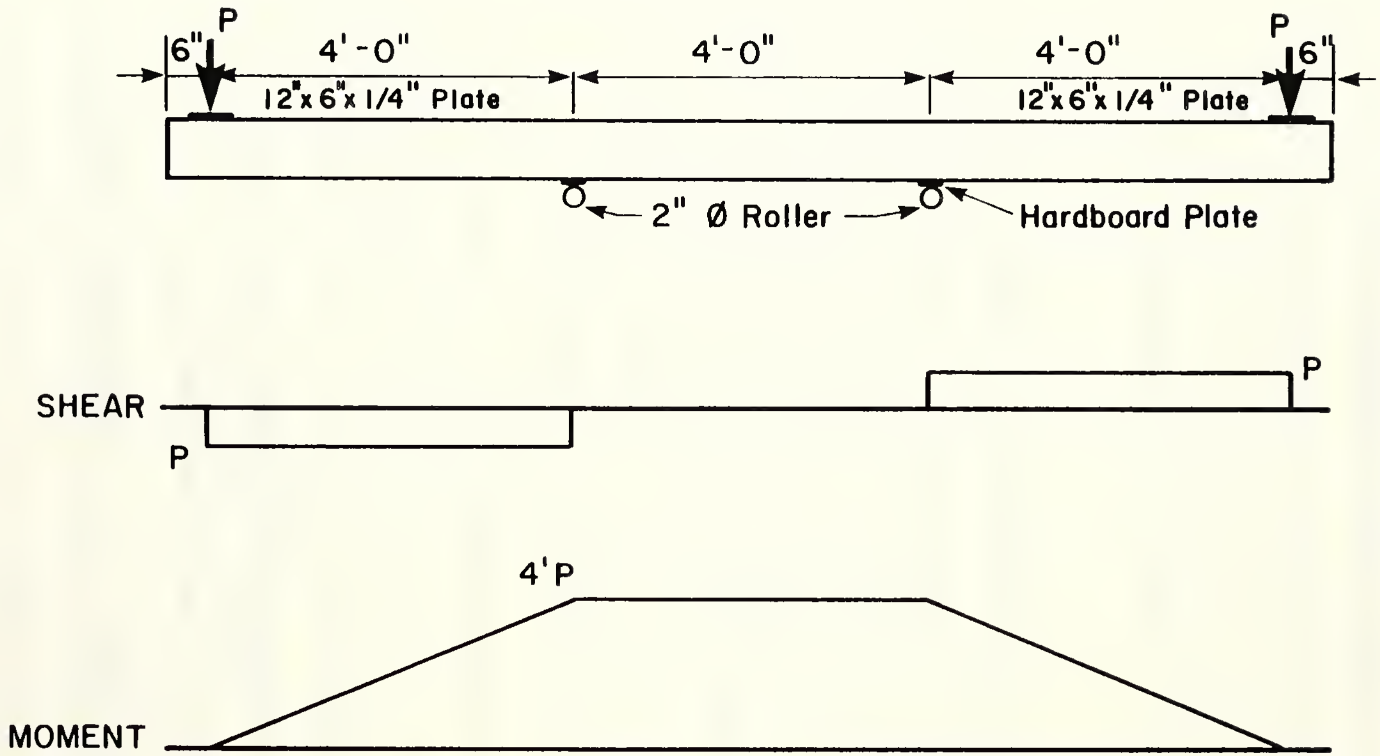
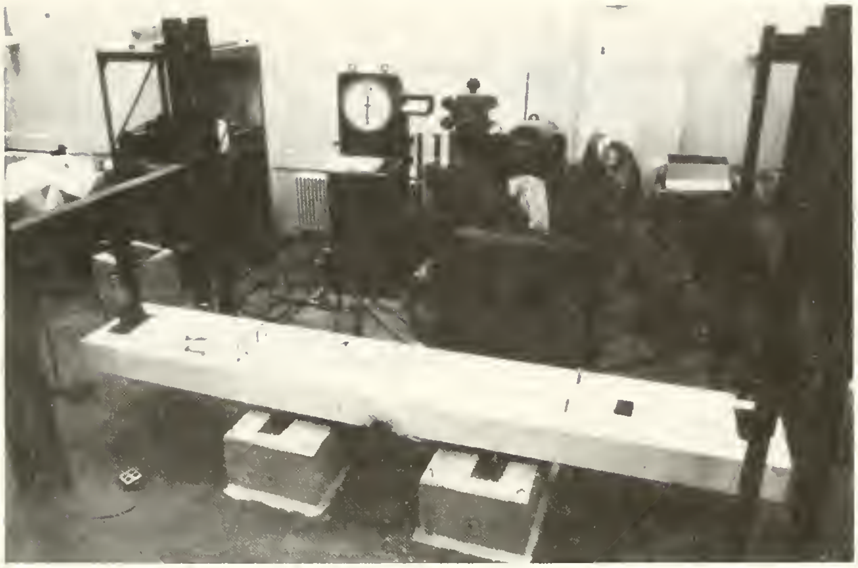


Figure 2.2 Slab Loading Arrangement.



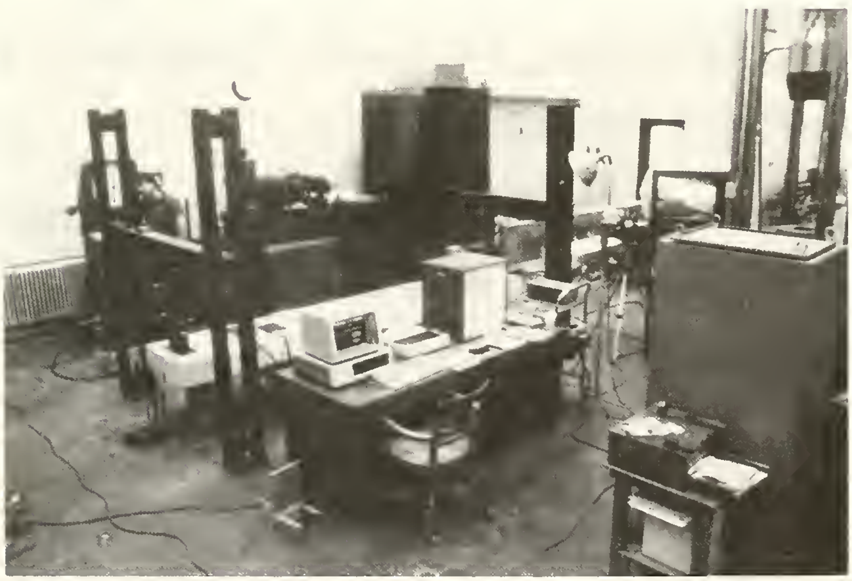


Figure 2.3 Slab in Position for Testing.

2.4 Materials

2.4.1 **Reinforcing Steel** Reinforcement consisted of #6 bars all from the same heat of steel to ensure that the coated bars and uncoated bars had identical deformations and mechanical properties. The bars had a spiral deformation pattern with a lug spacing of 22 lugs per 6 in. The deformations had a height of 0.045 inches. Table 2.1 summarizes the physical and mechanical properties of the reinforcing steel.

Epoxy coating thickness was measured with a Nordsen Dry Film Gage. Each bar was measured in three places along the longitudinal rib. Coating thickness had a mean of 9.0 mils with a standard deviation of 2.1 mils. This is in conformance with the Indiana Department of Highways Standards requiring a coating thickness of 6 to 12 mils[10]. The distribution of measured coating thicknesses is shown in Figure 2.4.

A mill test report was received with the reinforcing steel. The bars had a well-defined yield plateau at 65.2 ksi and an ultimate strength of 101 ksi. The values were confirmed with six coupon tests in the laboratory. A typical stress-strain curve is shown in Figure 2.5.

Table 2.1. Properties of Reinforcing Bars.

	Measured	ASTM
Maximum Gap (inches)	.125	.286 (max)
Maximum Spacing (inches)	.338	.525 (max)
Average Height (inches)	.045	.038 (min)
Variation in Weight (%)	5.2	6.0 (max)
Yield Strength (psi)	65,200	60,000 (min)
Tensile Strength (psi)	101,200	90,000 (min)
Elongation in 8 inches (%)	15.7	9.0 (min)

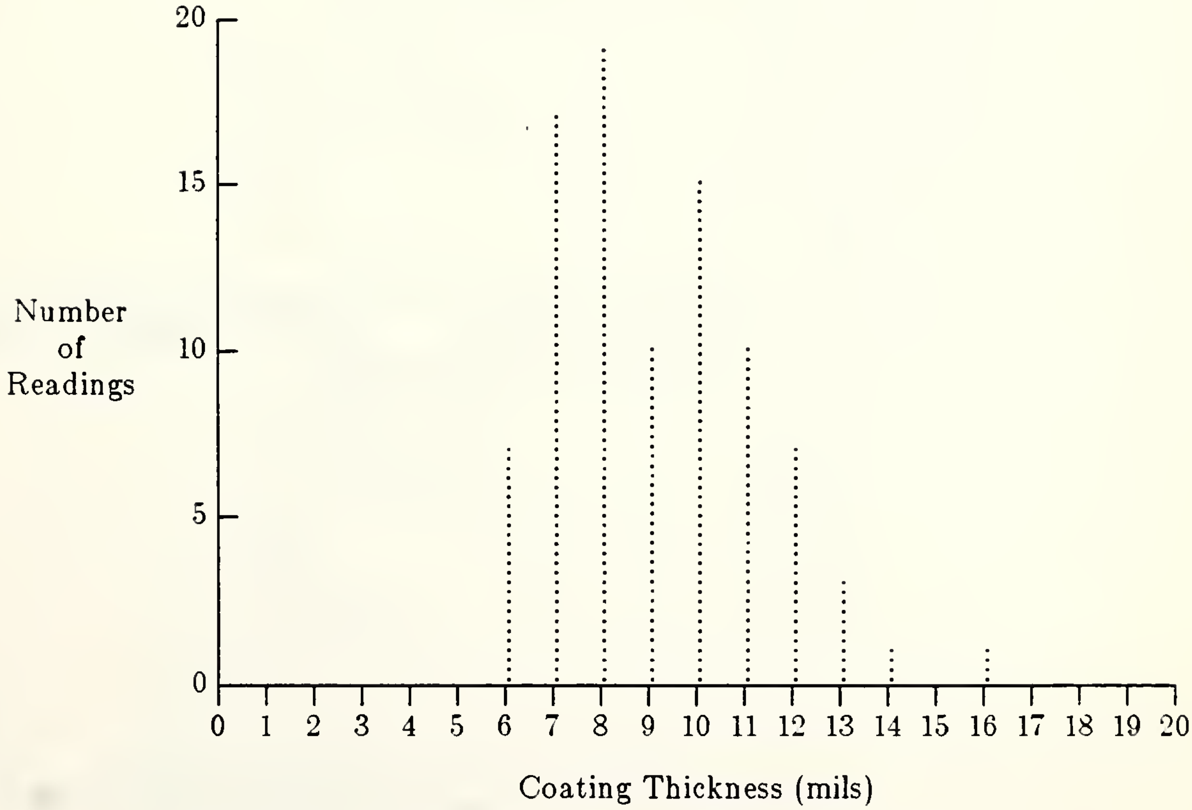


Figure 2.4. Distribution of Measured Coating Thicknesses.

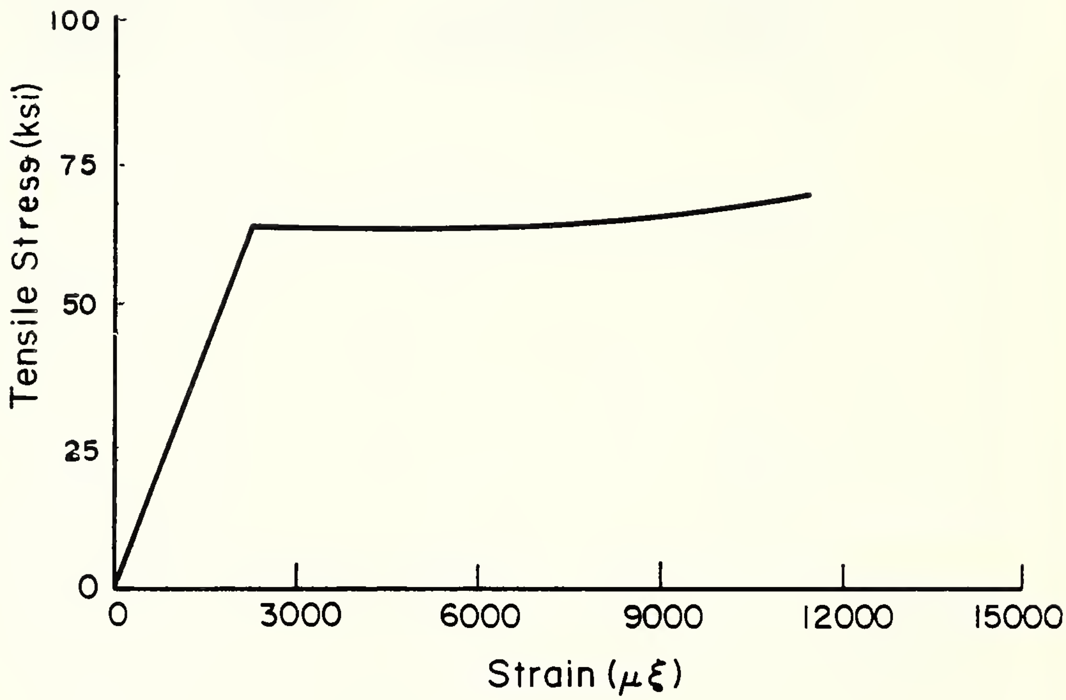


Figure 2.5 Steel Stress-Strain Curve.

2.4.2 **Concrete** Two concrete mixes were used. An air-entrained 5-bag mix with a nominal strength of 4000 psi was selected for the 16in., 14in.. and 12in. splice series. For the 10in. splice series a 9-bag mix with a nominal strength of 8,000 psi was used. This mix was not air-entrained. All concrete was batched at a local ready-mix plant and delivered to the casting site. The mix proportions per cubic yard are given below. Table 2.2 summarizes the concrete properties for each test series.

4000 psi Mix (28 Day Design Strength)	
Cement (Type I)	440 lb
Fly Ash (Class C)	100 lb
Gravel (IDOH #8)	1850 lb
Sand	1320 lb
Water	22 gal
Air entraining	6 oz

8000 psi Mix (28 Day Design Strength)	
Cement (Type I)	846 lb
Fly Ash (Class C)	250 lb
Gravel (IDOH #8)	1500 lb
Sand	1300 lb
Water	33 gal
Plasticizer	169 oz

2.5 Construction of Specimens

2.5.1 **Fabrication** Two identical forms were constructed so that both specimens of a series could be cast at once from the same batch of concrete. The longitudinal bars were spaced across the tops of four 5-1/4in. continuous

Table 2.2. Concrete Properties.

Series	Compressive Strength Test psi	Compressive Strength 28 Day psi	Tensile Strength (at testing) MOR psi	Tensile Strength (at testing) Splitting psi	Modulus of Elasticity (at testing) ksi
U16	5620(3)	5780(3)	645(3)	445(3)	4390(2)
E16	5520(3)	5780(3)	645(3)	445(3)	4390(2)
U14	5380(3)	6100(3)	610(3)	445(3)	4650(2)
E14	5840(3)	6100(3)	610(3)	445(3)	4700(2)
U12	3990(3)	4650(3)	560(2)	410(3)	4120(2)
E12	3990(3)	4650(3)	560(2)	410(3)	4120(2)
U10	8200(6)	8825(3)	613(3)	520(6)	4950(2)
E10	8200(6)	8825(3)	613(3)	520(6)	4950(2)

Number of tests indicated in parenthesis.

high chairs to maintain the correct cover and spacing of the bars. Proper alignment of the reinforcement was provided by a #4 bar cut to the width of the forms and tied to each of the two chairs closest to the beam end. The formwork prior to casting is shown in Figure 2.6.

2.5.2 **Casting** Both specimens of a series were cast simultaneously. The casting was done indoors with the concrete being placed directly from the ready-mix truck. (See Figure 2.7.)

The concrete was placed in a single lift and compacted with a mechanical vibrator. Slump and air content measurements were taken at this time. Flexure beams and cylinders were also made as the beams were cast.

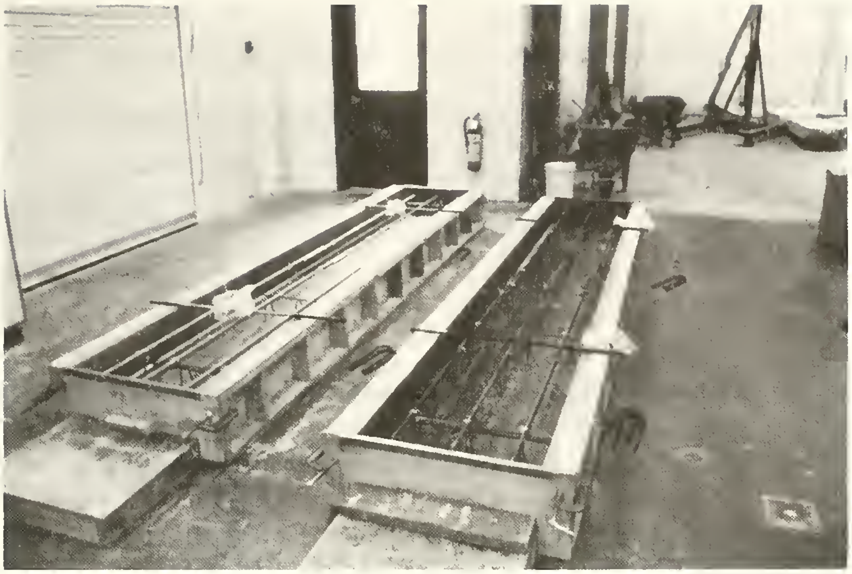


Figure 2.6 Slab Formwork.

Figure 2.7 Casting of Slabs.

After placing the concrete the beam surface was finished with trowels. The beams were then covered with wet burlap for 72 hours. The side forms were usually stripped 3 or 4 days after casting. The beams were left on the form bases until the test date.

2.6 Test Procedure

2.6.1 **Instrumentation** Load was measured with 10,000 pound capacity load cells at each end of the member and confirmed with readings from the load dial of the testing machine. End deflections were measured with linear variable differential transformers (LVDT's) mounted on the centerline of the longitudinal axis of the specimens. Dial gages were also placed at the ends to check the LVDT readings. LVDT's were mounted on each side at midspan of the members to record centerline deflections. These readings were also confirmed with dial gages.

The beams were whitewashed with a diluted latex paint to aid in crack detection. Crack widths were measured with crack width comparators.

Concrete strains were determined with 2in. gage surface strain gages mounted two per side in the flexural compression zone of the specimen. These gages were at 6-3/8in. and 7in. from the top concrete fiber. Each bar was instrumented in the constant moment region with a strain gage 14in. from the supports. All load cell, LVDT, and strain gage measurements were electronically recorded with a Data Acquisition system driven by a Personal

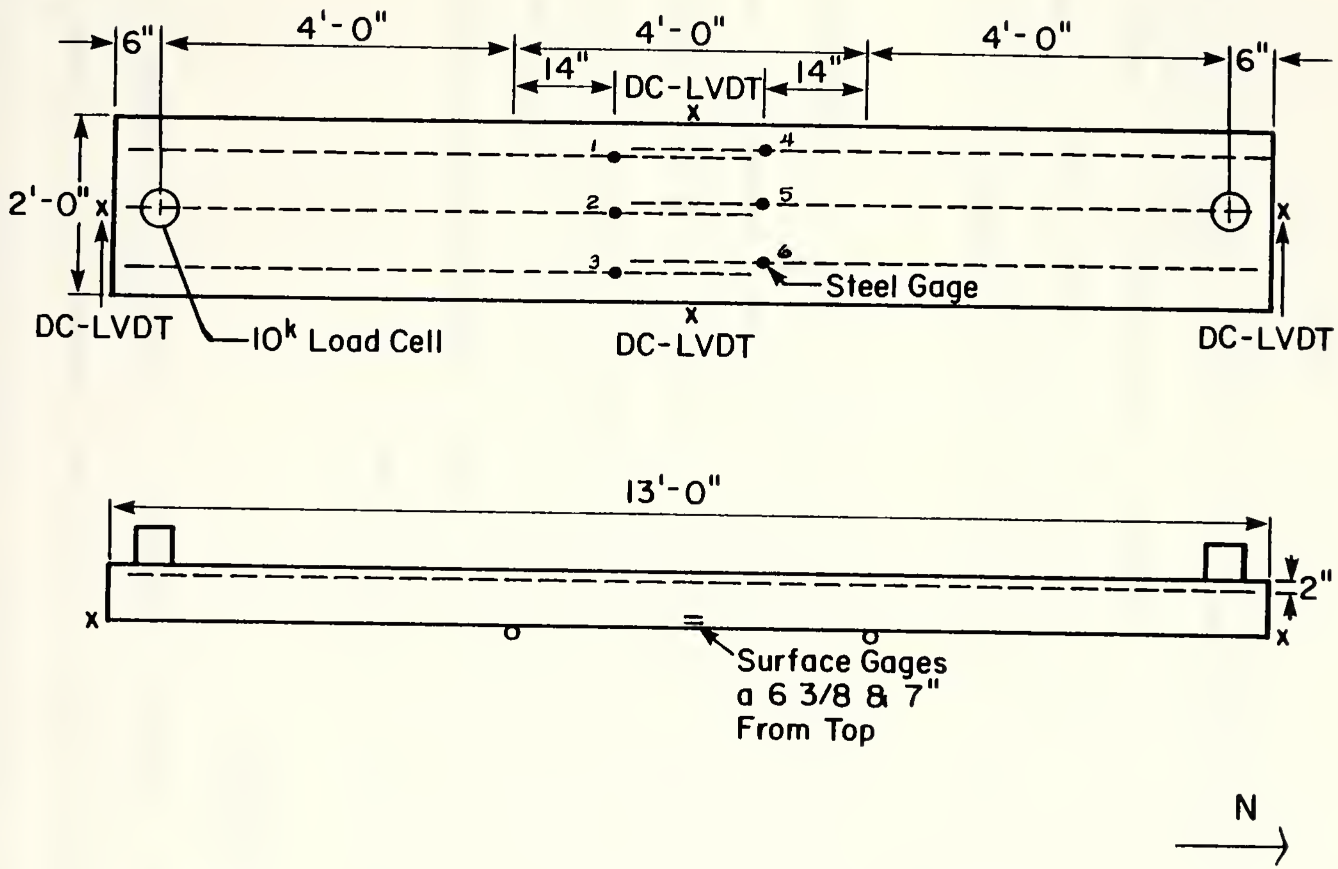


Figure 2.8 Test Instrumentation.

Computer. Figure 2.8 shows the location of all instrumentation on the test slabs.

2.6.2 **Loading Procedure** Loads were applied to the specimen through two 2-1/2in. bore hydraulic rams driven by an Amsler pendulum dynamometer. The load was gradually applied in 500 pound increments. At each increment the load was held while the dial gages were read and cracks marked.

During the periods the load was held steady, crack widths were measured using crack width comparators. These measurements were made to help evaluate service load behavior. In the 16in. and 12in. splice series these measurements were taken until failure. In the 14in. and 10in. splice series these measurements were terminated after 6 kips with the subsequent loads only held long enough to mark cracks. This was to eliminate the possibility of a failure during crack measurements.

2.7 Summary

The experimental program was developed to evaluate the ultimate strength and service load behavior of slabs reinforced with epoxy coated bars. For this purpose four series of identical specimens, one with uncoated steel and one with epoxy coated steel, were cast. The reinforcing steel consisted of three #6 bars spliced at the midspan. The specimens were loaded in an inverted third point loading. Bond stresses were evaluated over the length of the splices. Service load behavior was evaluated through comparisons of the deflections and crack widths and spacing between like epoxy coated and uncoated specimens. Chapter 3 presents the analysis of the results from the experimental program.

CHAPTER 3: ANALYSIS OF TEST RESULTS

3.1 Introduction

In this chapter the results of the four series of tests are presented. The performance of the splices is evaluated including cracking, deflections, and ultimate bond strength.

3.2 General Behavior

All of the tests resulted in splitting of the concrete at failure. Two occurrences during testing may have an effect on the reported data. In the test of slab U16 improper alignment of a loading ram caused the ram to displace. The subsequent release of energy resulted in reverse bending of the specimen. This stress reversal was indicated by a flexural crack through the entire depth of the slab. The reversal of loading may not have had an adverse effect on the bond; in the retest the steel reached yield just prior to failure. In test U14 there was substantial yielding of the steel prior to failure. At failure it appears a plastic hinge developed at the splice region.

3.3 Deflections

Load-deflection curves are shown in Appendix A, Figures A.1 through A.4. The load-deflection curve for both specimens of a series are shown on the same figure. From these figures it can be seen that there was no significant difference in the deflections due to the use of epoxy coated reinforcing steel. However, beams with epoxy coated bars showed a slightly larger deflection for a given load.

3.4 Cracking

3.4.1 **Flexural Cracking** First flexural cracking for all specimens occurred in the constant moment region outside of the splice length. The location of first cracking outside of the splice appeared to be random.

As seen in the load deflection plots of Appendix A (Figures A.1 through A.4) and Table 3.1, as expected, there were no significant differences in the load at first cracking between specimens with coated and uncoated bars.

Appendix B contains the crack width and spacing data. Figures B.1 through B.4 present the average crack width versus the load for each slab series. The data for both specimens of each series are presented on the same plot. These plots show wider average crack widths in the epoxy coated specimens than in the uncoated specimens, especially for the shorter splice lengths. This difference is especially clear with average loads greater than three kips.

Figure B.5 shows comparisons of the crack patterns at failure for each test series. In this figure the constant moment region is indicated by solid vertical lines and the splice region is shown by dashed vertical lines. All of the flexural cracks seen in the constant moment region were present at an average end load of 3 kips. This load is 30% to 50% of the ultimate load. This indicates these cracks occurred in what can be considered service load conditions. At failure the uncoated specimens of the 16in. series had seven flexural cracks in the constant moment region while the epoxy coated specimens had five. Both specimens of the 14in. series had seven flexural cracks in the constant moment region. In the 12in. series the uncoated specimen had seven flexural cracks in the constant moment region while the epoxy coated specimen had six. In the 10in. series the uncoated specimen had six cracks in the constant moment region and the epoxy coated specimen had five.

3.4.2 **Longitudinal Cracking** First longitudinal or splitting cracking occurred over the outer bars in the splice region. These cracks tended to initiate at a flexural crack. In the uncoated specimens there was a significant period of load increase after first splitting to the failure load in all but the 10in. series. In the epoxy specimens there was little load increase following first splitting in the 16in. and 12in. series. A load gain equivalent to that of the uncoated bars was seen in epoxy tests of the 14in. and 10in. series although there was a loss in ultimate strength in all tests. In the 14in. and

10in. series the load was not held for crack measurements after 6 kips. Splitting and ultimate loads are given in Table 3.1.

3.4.3 **Appearance After Test** After testing the slabs were split in half to reveal the reinforcing bars and concrete at the concrete-steel interface. The uncoated bars show a large amount of concrete adhering to them. The grooves left in the concrete by the bars are dull and worn in appearance.

The epoxy bars show no concrete adherence. The grooves left in the concrete are shiny and smooth. This indicates very little adhesive bond occurs with epoxy coated reinforcing steel. (See Figure 3.1 through 3.3).

Table 3.1. Cracking and Ultimate Loads.

	Average End Load (kips)		
Test	1st Flexural Crack	1st Splitting Crack	Ultimate Load
U16	2.0	7.5	-
U16*	-	-	9.0
E16	1.9	7.5	7.8
U14	2.0	6.0	8.3
E14	1.9	7.0	7.5
U12	1.9	6.0	6.5
E12	1.7	6.0	6.2
U10	1.8	8.0	8.3
E10	2.0	6.5	7.0

* Retest of U16

3.5 Discussion

3.5.1 **Serviceability Considerations** A primary purpose of this study was the evaluation of the behavior of bridge deck slab specimens reinforced

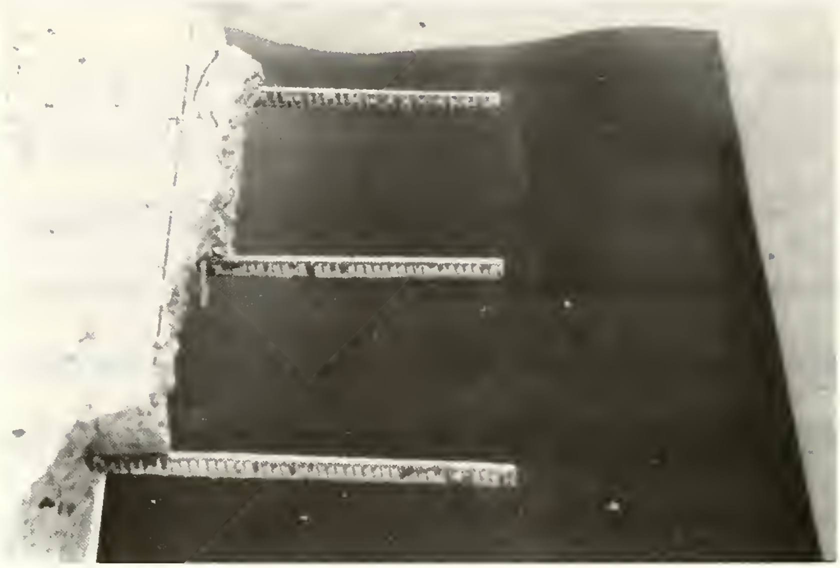


Figure 3.1 Uncoated Steel After Test.

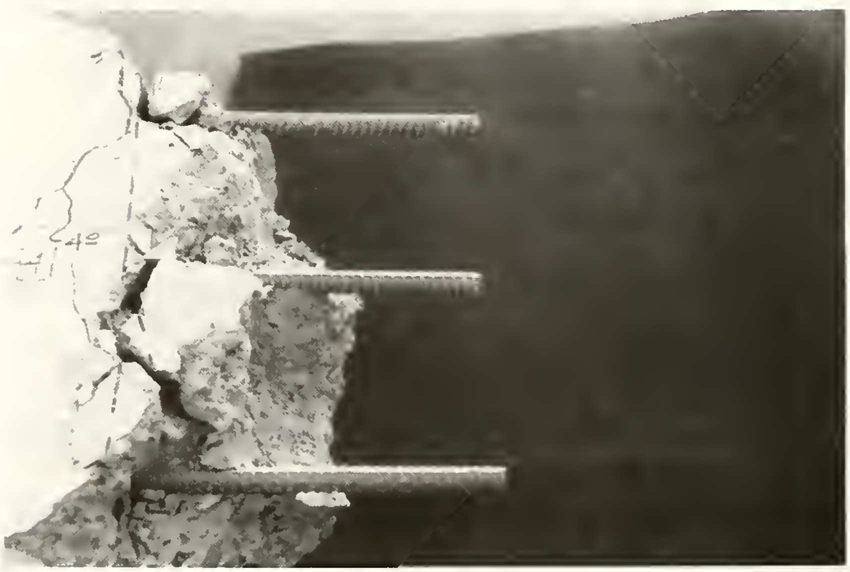


Figure 3.2 Epoxy Coated Steel After Test.

with epoxy coated bars under service load conditions. Important serviceability considerations include deflections and the number and size of flexural cracks.

Figure 3.3 Concrete After Test.

3.5.1.1 **Deflections** The stiffness of slabs with epoxy coated bars was compared to the stiffness of slabs with uncoated bars by plotting the average end deflection versus the average end load for each series. Each plot includes the load-deflection curve for both slabs of the series.

The load-deflection curves presented in Appendix A, Figures A.1 through A.4, reveal little difference in the stiffness of slabs reinforced with epoxy coated steel relative to those reinforced with uncoated steel. This was also observed by Treece and Jirsa[8].

3.5.1.2 **Crack Width and Spacing** The width and number of cracks in a bridge deck will have a significant effect on the concentration of chloride ions at the level of the reinforcing steel. Larger or more frequent cracking will increase the possibility of a concentration of chloride ions.

Figure B.5 in Appendix B presents the crack pattern of each slab at failure. In the 16in., 12in., and 10in. a reduction in the number of flexural cracks was seen between the uncoated and coated specimens in the constant moment region. Treece and Jirsa[8] also observed fewer cracks in epoxy specimens.

In order to compare crack widths between the epoxy specimens and uncoated specimens the average crack width of the three largest cracks which occurred in each slab were plotted versus the load. Figures B.1 through B.4 of Appendix B show wider average crack widths for members with epoxy coated reinforcement.

3.5.1.3 **Influence of Epoxy Coating on Serviceability** Use of epoxy coated reinforcing steel will have no adverse effects on the serviceability of bridge deck members. The tests reveal no loss of stiffness due to epoxy coatings. Although crack widths are wider, there are fewer cracks when epoxy coatings are used. If the coating is continuous, epoxy coating will prevent corrosion due to chlorides which may reach the reinforcement due to wider cracks.

3.5.2 **Evaluation of Bond Strength** In each test the mode of failure was a splitting failure over the spliced bars. This is an indication that the splice had reached its maximum capacity. For this reason the bond strength developed over the length of the splice in the splitting mode of failure can be evaluated.

The bond stress is defined as the axial force developed in the reinforcing bar distributed over the surface area of the bar.

$$u = \frac{A_b f_s}{\pi d_b l_d}$$

The axial stress developed in the bars was determined with the measured strains from strain gages placed on the bar surface in the constant moment region but outside of the splice. Strain readings on the bars were confirmed by the strain measurements taken in the concrete compression zone assuming a linear strain distribution. These readings are shown in Appendix C.

Two empirical formulas for bond stress were presented in Section 1.2,

$$u = 9.5\frac{\sqrt{f_c'}}{d_b} \qquad \text{(u-ACI)}$$

which forms the basis for the ACI 318–83[1] provisions for basic development length, and

$$u = (1.2 + \frac{3C}{d_b} + 50\frac{d_b}{l_d})\sqrt{f'_c} \qquad \text{(u-Orangun)}$$

developed by Orangun, Jirsa, and Breen[2] for members with no transverse reinforcement.

Treece and Jirsa[8] introduced a parameter defined as the bond efficiency. The bond efficiency is the ratio of the experimentally determined bond stress to the bond stress predicted by an empirical or theoretical expression. Table 3.2 summarizes the results of the eight slab tests. The bond efficiency is measured relative to the theoretical values of the ACI, and the Orangun, Jirsa, and Breen proposed expression. The values given for the bond efficiency of the epoxy bars relative to the empirical equations may not give a meaningful measure of bond reduction. They do, however, show that these expressions cannot predict the bond stress for epoxy bars.

The reinforcing steel yielded in slabs U16 and U14. This produces artificially low values for bond efficiency as, after yield, the steel cannot develop the level of bond stress predicted by the empirical expressions. At yield the bar force does not increase with increasing deformation of the steel. Because the bond stress has been defined as this force over the surface area of the bar, bond stress becomes a constant at yield.

Column 7 of Table 3.2 indicates that the equation designated as u-Orangun is a good predictor of bond stress for the uncoated bars. Neglecting U16 due to yielding, all values are within 2% of the predicted value. Column

6 indicates that the ACI formula is reasonably accurate for these tests with all values within 6%.

The table includes the bond ratio which is the ratio of the normalized bond stress developed in the epoxy coated bars to that developed in the uncoated bars. In order to normalize for differences in concrete strength the bond stress in each test was divided by $\sqrt{f'_c}$. These values were then compared to give the bond ratios presented in Table 3.2.

Although included in Table 3.2, the bond ratios for the 16in. and 14in. splice series will not be used in further qualitative evaluations of bond strength due to the steel yielding in the uncoated specimens. They do however show the expected decrease in bond strength due to epoxy coating.

The test results show a reduction in bond strength due to the epoxy coating. For the two series which resulted in bond failures prior to steel yielding there is a large difference in the measured bond ratios. The bond ratio for the 12" series is 0.97. The bond ratio for the 10" series is 0.65.

Two factors may have lowered the bond ratio in the 10" series. These factors are the concrete strength and the number of flexural cracks crossing the splice.

Forces are transferred between the concrete and steel through rib bearing, friction, and adhesion. The bearing mechanism produces radial stresses which can lead to a splitting failure. However, there must be some bar slip for the bearing mechanism to act. This slip is greatest at crack locations and reduced as one moves away from the crack. Therefore the bearing force and thus the splitting stress are greatest at crack locations[12].

The magnitude of the bearing force is controlled by the amount of slip which depends on the degree to which adhesion and friction forces are overcome. As was seen, concrete shows good adhesion to uncoated bars. In order to overcome the adhesive force the concrete along the bar must be split or sheared allowing the bar to move relative to the concrete. Thus higher strength concretes may have a larger bond contribution from adhesion and friction than do lower strength concretes.

Epoxy coated bars show a reduced adhesion to concrete implying the adhesion and friction contributions are smaller than in uncoated bars. This will increase slip and bearing leading to fewer and wider cracks. Increased slip and bearing in conjunction with little adhesive force would cause a greater reduction in bond strength with epoxy coated bars relative to uncoated bars when used in high strength concrete.

The second factor, the number of flexural cracks may also be important in the bond reduction seen in the 10" series. As seen in Figure B.8 in Appendix B, the uncoated specimen had three flexural cracks in the splice region whereas only two cracks were in this region of the epoxy specimen. Because slip and bearing are greatest at crack locations, a large portion of the bar force is transferred at these points. Because this portion of the bar force is distributed over three points in the uncoated specimen compared to only two points in the epoxy coated specimen, in addition to the increased bearing due to lack of adhesion, a large bond strength reduction could be expected for this series in the coated bars.

The 12in. splice series shows the trend of higher bond ratios for lower strength concretes. This series had a compressive strength of 3990 psi. Increases in slip in this series may be indicated by larger deflections seen in this series at a given load relative to those in other series. (Figure A.3.)

Crack location may have also influenced the results of this series. As seen in Figure B.5 of Appendix B, the flexural cracks in the splice region of the uncoated specimen were located very close to the ends of the splice whereas they were more central in the epoxy coated specimen.

The transfer of stress from the reinforcing bar to the concrete is considered to be larger at the loaded or continuous end of the splice and very low at the unloaded end. In the uncoated specimen the majority of the force transfer for each bar will be concentrated at the loaded end crack. In the epoxy specimen the force transferred will be relatively better distributed between the two cracks. This factor may have helped to overcome the loss in bond due to the lack of adhesive forces in the epoxy specimens.

Table 3.2. Results of Slab Tests.

Test	f_s (ksi)	u-test (psi)	u-ACI (psi)	u-Orangun (psi)	u-test / u-ACI	u-test / u-Orangun	Bond Ratio
U16	65.2*	761	950	865	.801	.880	
E16	58.8	686	941	858	.729	.800	.91
U14	65.2*	870	929	871	.937	.999	
E14	53.6	715	968	908	.759	.787	.79
U12	49.0	763	800	779	.954	.979	
E12	47.3	736	800	779	.920	.945	.97
U10	63.5	1186	1147	1173	1.034	1.011	
E10	41.5	775	1147	1173	.676	.661	.65

*yield strength

3.5.2.1 **Influence of Concrete Strength and Splice Length on Bond Ratio** The results from the tests of Johnston and Zia and those of Treece and Jirsa have been included to further evaluate the influence of concrete strength and splice length on bond ratio. These results, normalized as described in Section 3.5.2, are shown in Table 3.3. In this table the tests of Treece and Jirsa are indicated by a three digit code such as 12-6-4 where the first digit gives the coating thickness in mils, the second digit gives the bar size and the third digit gives the approximate concrete strength. Those of Johnston and Zia are indicated by a code such as BS-11M-16. The first digit indicates a beam series, the second gives bar size and coating (mill scale or epoxy) and the third is development length. These results only include series in which the reinforcing steel did not yield and those grouped around common concrete strengths.

Table 3.3. Results of Slab and Beam Tests.

Test	$\frac{C}{d_b}$	l_d (inches)	f'_c (psi)	u-test (psi)	Bond Ratio
U12	2.67	12	3990	763	-
E12	2.67	12	3990	736	.97
U10	2.67	10	8200	1186	-
E10	2.67	10	8200	775	.65
0-6-4	2.67	12	4250	830	-
5-6-4	2.67	12	4250	722	.87
12-6-4	2.67	12	4250	516	.62
0-6-4r	1.33	24	3860	495	-
5-6-4r	1.00	24	3860	374	.76
12-6-4r	1.17	24	3860	350	.71
0-11-4b	1.45	36	4290	449	-
12-11-4b	1.45	36	4290	244	.54
0-6-8	1.17	16	8040	742	-
12-6-8	1.00	16	8040	410	.55
0-11-8	1.55	18	8280	789	-
12-11-8	1.64	18	8280	495	.63
BS-11M-16	1.46	16	6480	1005	-
BS-11E-16	1.46	16	6562	793	.78
BS-11M-24	1.46	24	6480	776	-
BS-11E-24	1.46	24	6562	543	.70
BS-11M-30	1.46	30	6480	621	-
BS-11E-30	1.46	30	6562	395	.63

Figure 3.4 is a plot of bond ratio versus splice length. The data has been plotted for concrete compressive strengths grouped around 4100 psi, 6600 psi, and 8200 psi. The concrete strengths in any group do not vary by more than 500 psi from the group mean. Two trends are seen in this figure. The first, a reduction in bond ratio with increasing concrete strength was discussed in Section 3.5.2. The second trend seen is a reduction in bond ratio with increasing splice length. As noted in Section 3.5.1.2, there is a tendency

toward fewer flexural cracks in members with epoxy coated reinforcing steel relative to those with uncoated steel. As splice length increases the possibility of an uncoated specimen having more cracks across the splice than the epoxy coated specimen increases. This may have the effect of reducing the bond ratio by placing fewer but larger stress concentrations on the concrete as discussed in Section 3.5.2. A longer splice length also allows the friction and adhesion mechanism of bond to act over a larger area of the uncoated bars. This may be the most important factor in the increased bond reduction seen with increasing splice length.

Figure 3.4 shows an approximate bond strength reduction of 1% for each inch of anchorage for the 4100 psi and 6600 psi concretes. The reduction is approximately 0.6% for each inch of anchorage in the 8200 psi concretes. The points considered in Figure 3.4 for the concrete strength of 6600 psi contained transverse reinforcement. This may have had the effect of overcoming the reduction due to higher concrete strength pushing the line close to that for the 4100 psi concrete.

It may be important to note that differences in the cover to bar diameter ratio have not been accounted for in this analysis. Because equal reductions in bond ratio can be seen for all values of the cover to bar diameter, it is not considered to have a significant influence on the bond ratio for values of cover to bar diameter less than three. However, the cover to bar diameter ratio has not been truly normalized. Indeed, splitting may become critical in epoxy bars at a cover to bar diameter ratio greater than three.

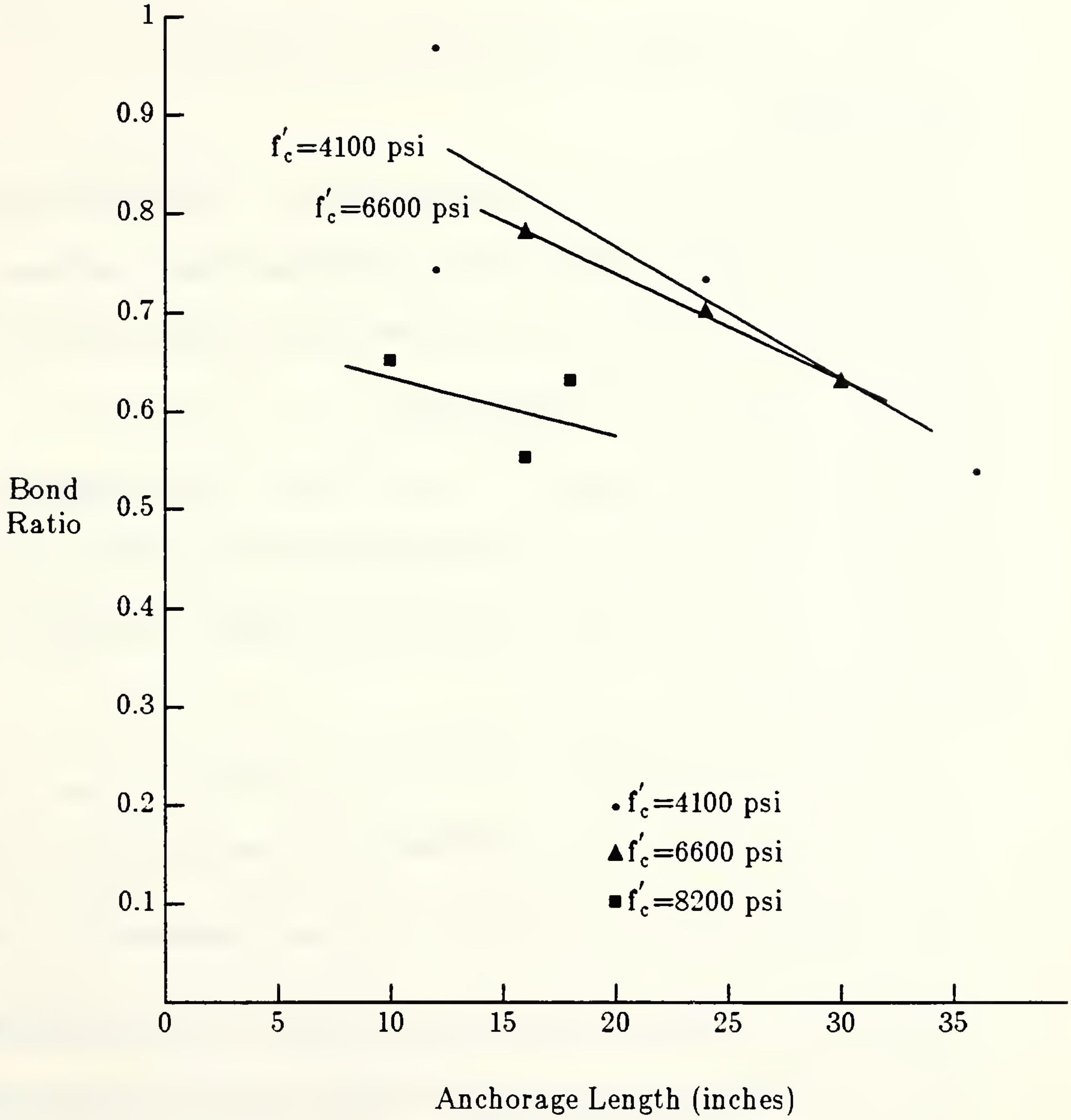


Figure 3.4. Bond Ratio versus Anchorage Length.

3.5.2.2 **Influence of Epoxy Coating on Bond Strength** Epoxy coating of reinforcing steel has an adverse effect on bond and ultimate strength. The reduction in bond does not seem to be more significant in the slab specimens than in previously tested beam specimens. In fact the slab tests contained bond ratios close to the high and low extremes of all tests.

3.6 Summary

The use of epoxy coated reinforcing steel has no adverse effects on the serviceability of slab members. No loss of stiffness relative to members with uncoated steel has been seen. Although the slabs with epoxy coated steel had wider crack widths, there were fewer cracks.

Epoxy coatings do reduce the ultimate bond stress developed by the reinforcing steel. However this reduction does not appear to be a function of specimen type. The slab tests show the same degree of reduction seen in previous beam tests.

The bond ratio, the ratio of bond stress developed in epoxy coated specimens relative to the bond stress developed in similar uncoated specimens for a given $\frac{c}{d_b}$ ratio, appears to be strongly influenced by concrete compressive or splitting strength and splice length. There is a reduction in bond ratio with increasing concrete strength possibly due to an increased adhesive component of the bond mechanism with the uncoated steel. There is also a reduction in bond ratio with increasing length of anchorage. This may be caused by a larger area over which the adhesion of the uncoated bars may act and due to the possible presence of fewer cracks in the epoxy coated specimens. Fewer cracks produce a greater radial stress concentration at each crack.

CHAPTER 4: SUMMARY AND CONCLUSIONS

4.1 Summary

Epoxy coated bars failing as a result of splitting the concrete cover show a reduction in the ultimate bond strength. Based on the results of this and two previous studies[7,8] there does not appear to be a significant difference in the behavior of slabs with epoxy coated bars relative to beams with epoxy coated bars without transverse steel failing due to splitting of the concrete cover. Transverse steel may diminish the influence of the epoxy coating in reducing bond although this cannot be verified with the limited available number of tests with transverse steel resulting in splitting failures.

Treece and Jirsa[8] recommended an increase of 50% in the basic development length for epoxy bars where splitting would be the controlling mode of failure ($C<3d_b$). For all other cases a 15% increase was recommended. These recommendations were based on an average ratio of epoxy coated bar bond stress to uncoated bar bond stress (bond ratio) of 0.66. The reciprocal of this gives the factor of 1.50. The ACI has adopted this factor for the 1989 Edition of the Building Code[14].

In the development of this recommendation the effects of concrete strength and provided anchorage length did not seem to be properly accounted for.

Using the current tests and those of the two previous studies[7,8] an influence of both concrete strength and anchorage length was seen on the ratio of bond between coated and uncoated bars. The reduction in bond due to epoxy coating appears to increase with increasing anchorage and increasing concrete strength. Thus the previously defined bond ratio decreases with increasing anchorage and increasing concrete strength. The reduction in bond ratio due to increasing anchorage may result from wider crack spacing and thus fewer cracks in specimens with epoxy bars, causing greater stress concentrations at each individual crack. In addition, increased anchorage length provides an increased area over which the frictional and adhesive forces may act in the uncoated specimens. The reduction in bond ratio due to increasing concrete strength may be due to an increased adhesion or reduced slip with uncoated bars and increasing concrete strength. The adhesion mechanism of bond is not effective with epoxy coated bars.

4.2 Further Research

This and previous studies have improved the basic understanding of the effect of epoxy coating on bond strength. However several questions still remain.

1. At what ratio of cover to bar diameter does a pullout failure occur with epoxy coated bars?
2. With what level of transverse reinforcement does a pullout failure occur and how does epoxy coating of both the longitudinal and transverse steel effect this level?

3. What is the influence of transverse steel on the bond ratio?
4. What is the influence of cyclic loading on bridge deck members reinforced with epoxy coated reinforcing steel?
5. What are the relative magnitudes of the bearing and adhesive forces in all types of reinforcing?

Further research should be directed at a behavioral model to better represent the bond mechanism. Shortcomings of the current empirical approach include the fact that the findings are limited to the tests run and must be extrapolated to other situations.

4.3 Conclusions

Based on the results of these slab tests along with data from previous studies the following conclusions can be made:

1. Epoxy coating significantly reduced the bond strength of reinforcing bars in tension. The amount of the reduction was dependent on the mode of failure, concrete strength, and anchorage length.
2. The bond strength reduction due to epoxy coating was especially significant for splitting failures.
3. There were fewer, but wider flexural cracks with epoxy coated bars.
4. Cracking load and deflections were not significantly affected by epoxy coating.
5. There was no observed difference in the behavior of slabs with epoxy coated bars relative to beams with epoxy coated bars without transverse steel.

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Appendix A

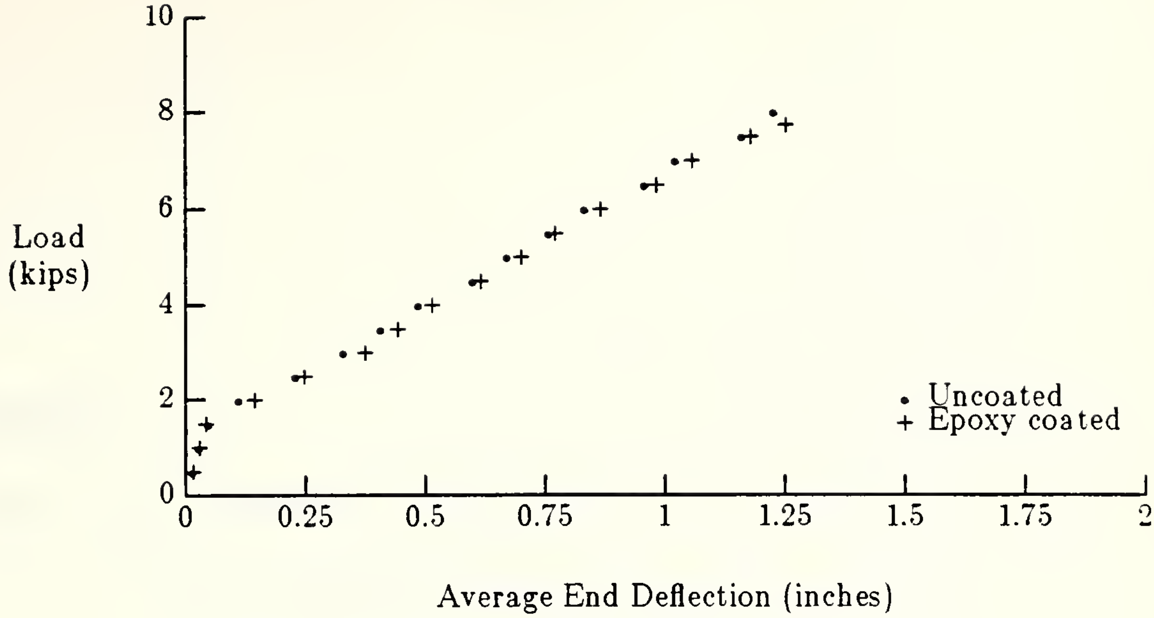

Figure A.1. Load-Deflection Curves, 16 in. Series.

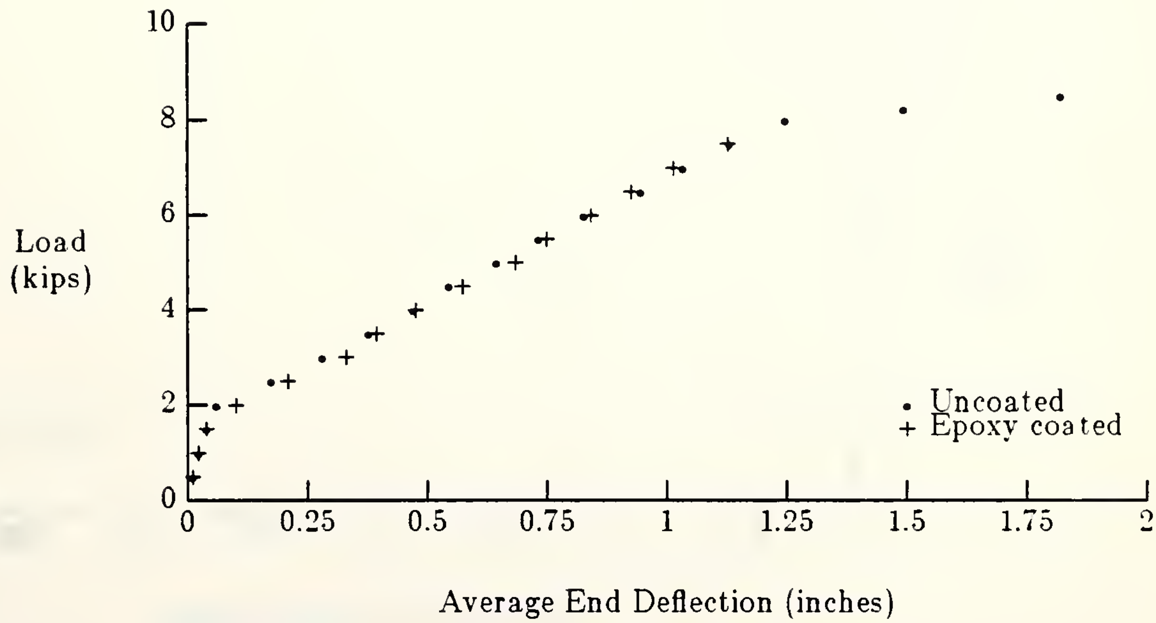

Figure A.2. Load-Deflection Curves, 14 in. series.

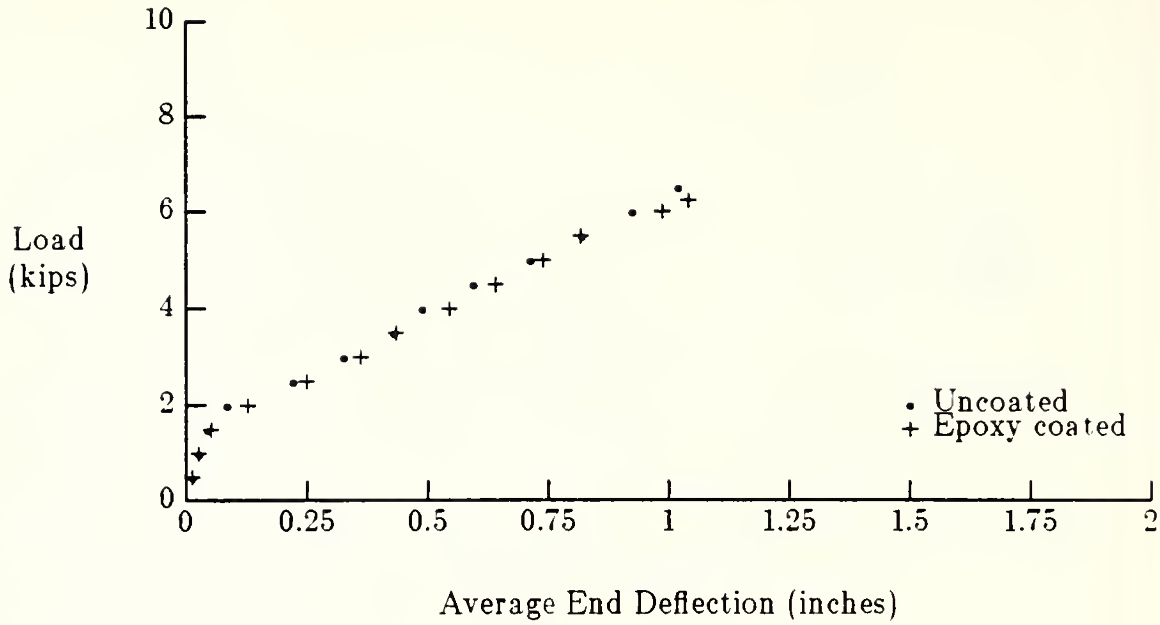

Figure A.3. Load-Deflection Curves, 12 in. series.

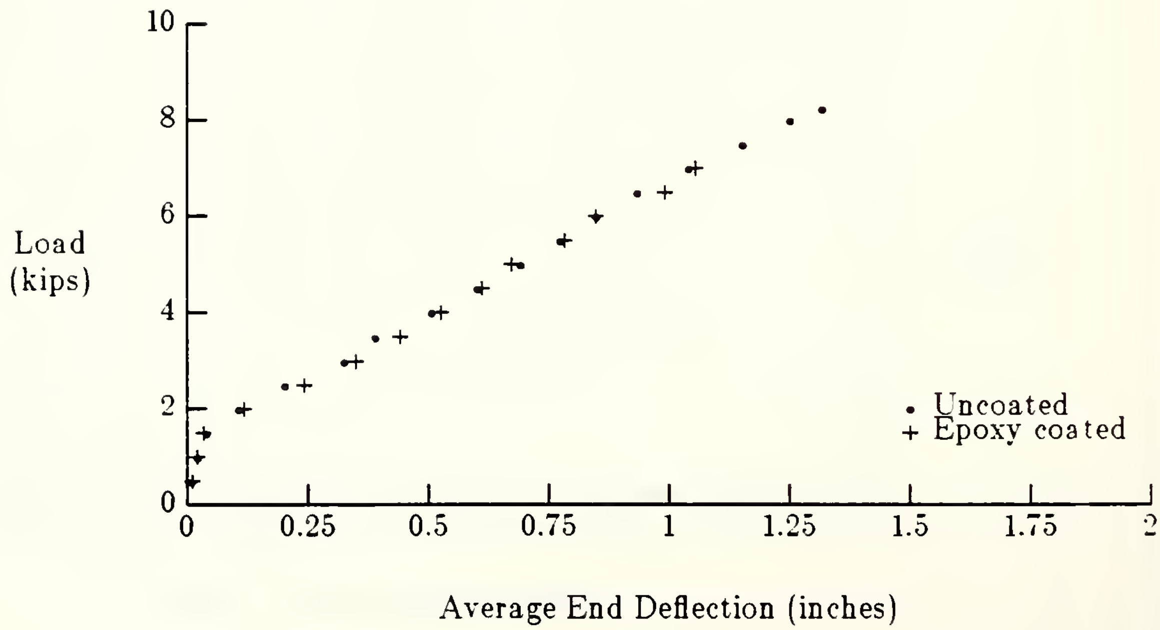

Figure A.4. Load-Deflection Curves, 10 in. series.

Appendix B

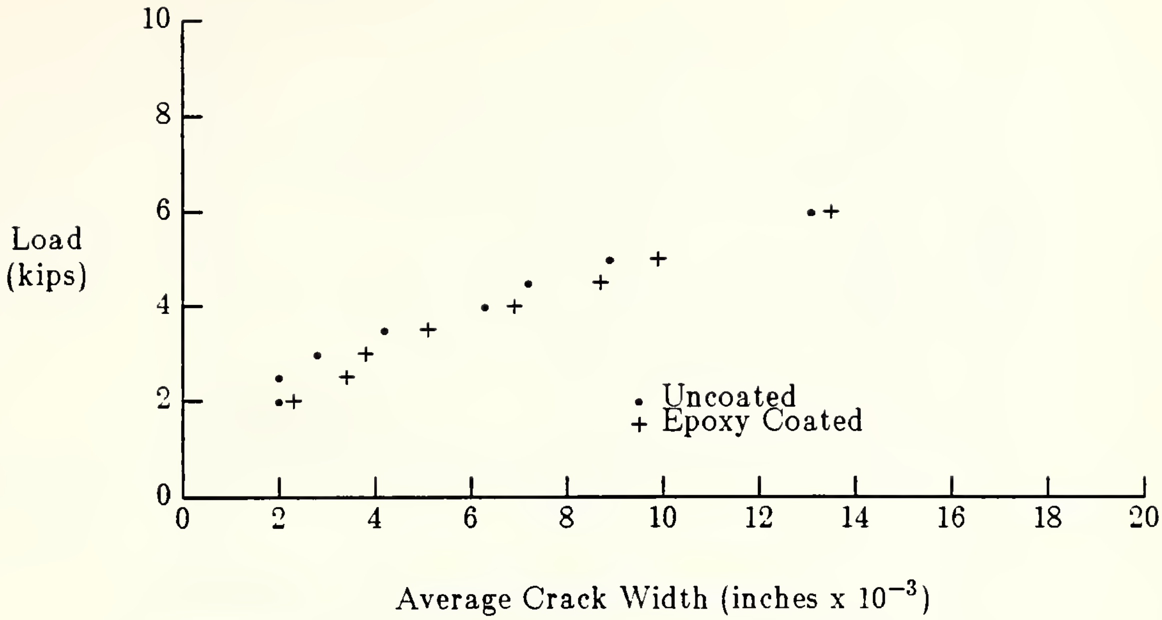

Figure B.1. Load versus Average Crack Width, 16 in. series.

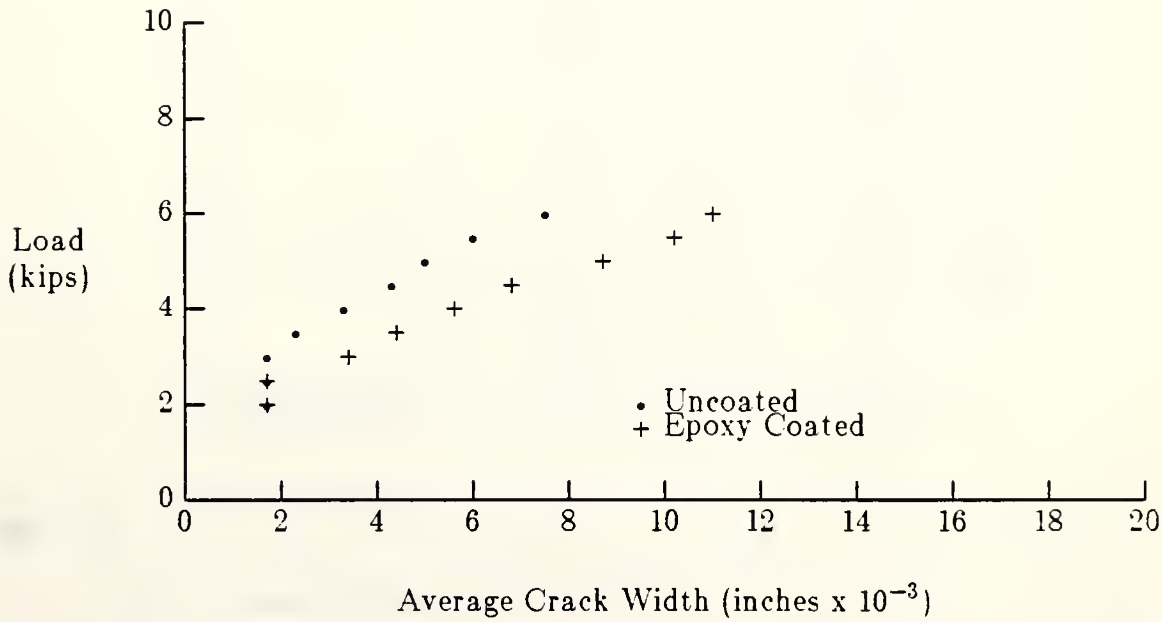

Figure B.2. Load versus Average Crack Width, 14 in. series.

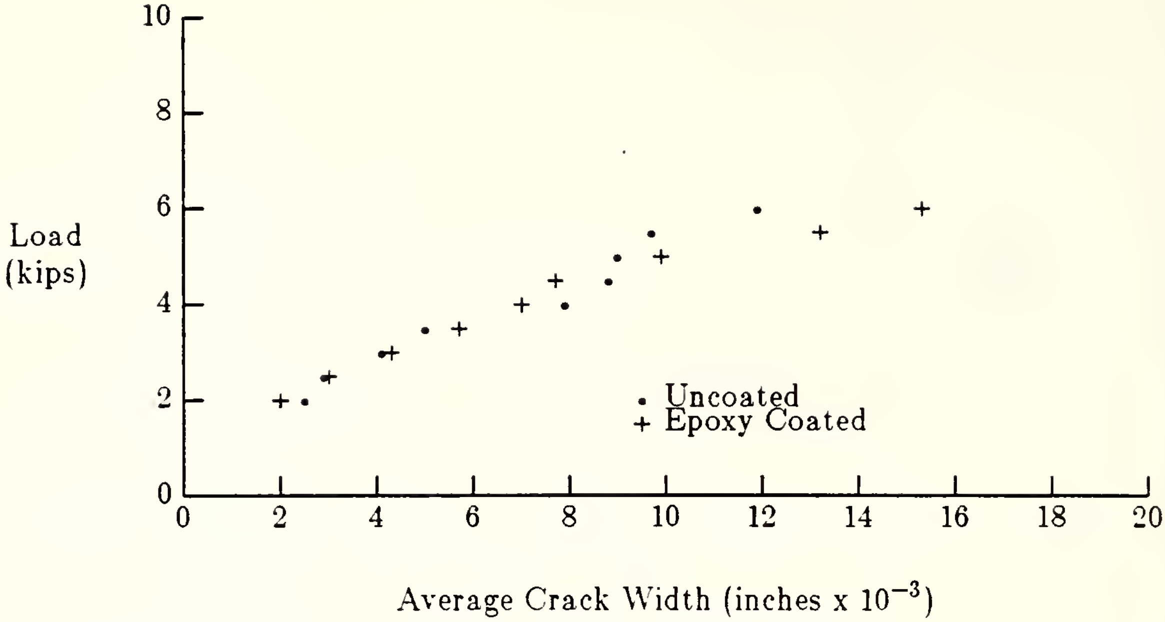

Figure B.3. Load versus Average Crack Width, 12 in. series.

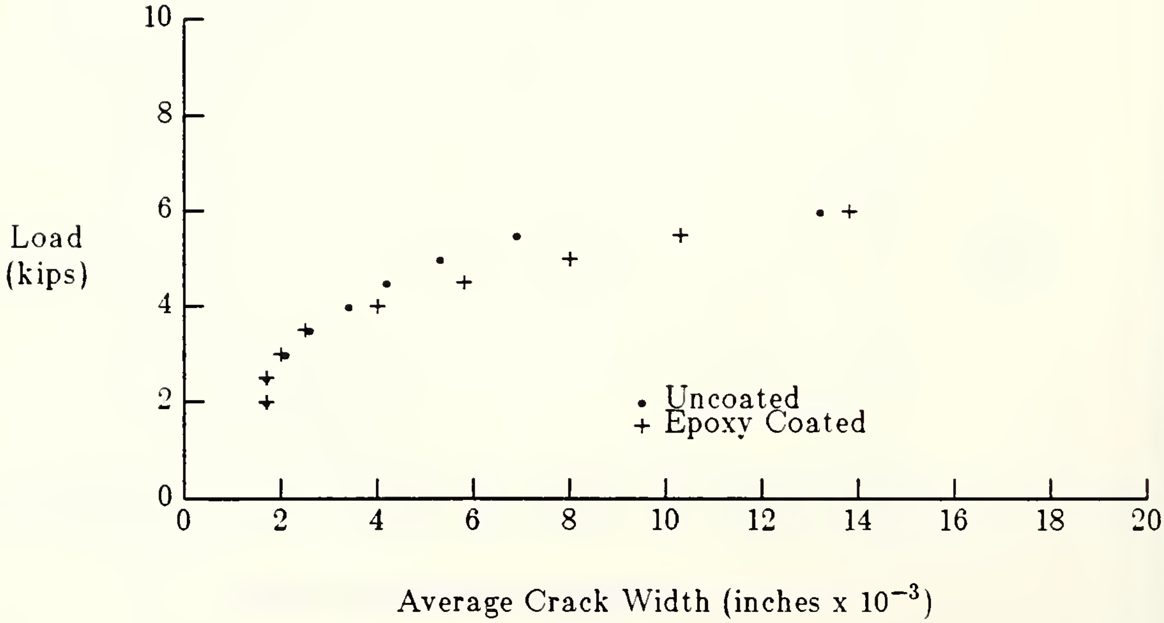

Figure B.4. Load versus Average Crack Width, 10 in. series.

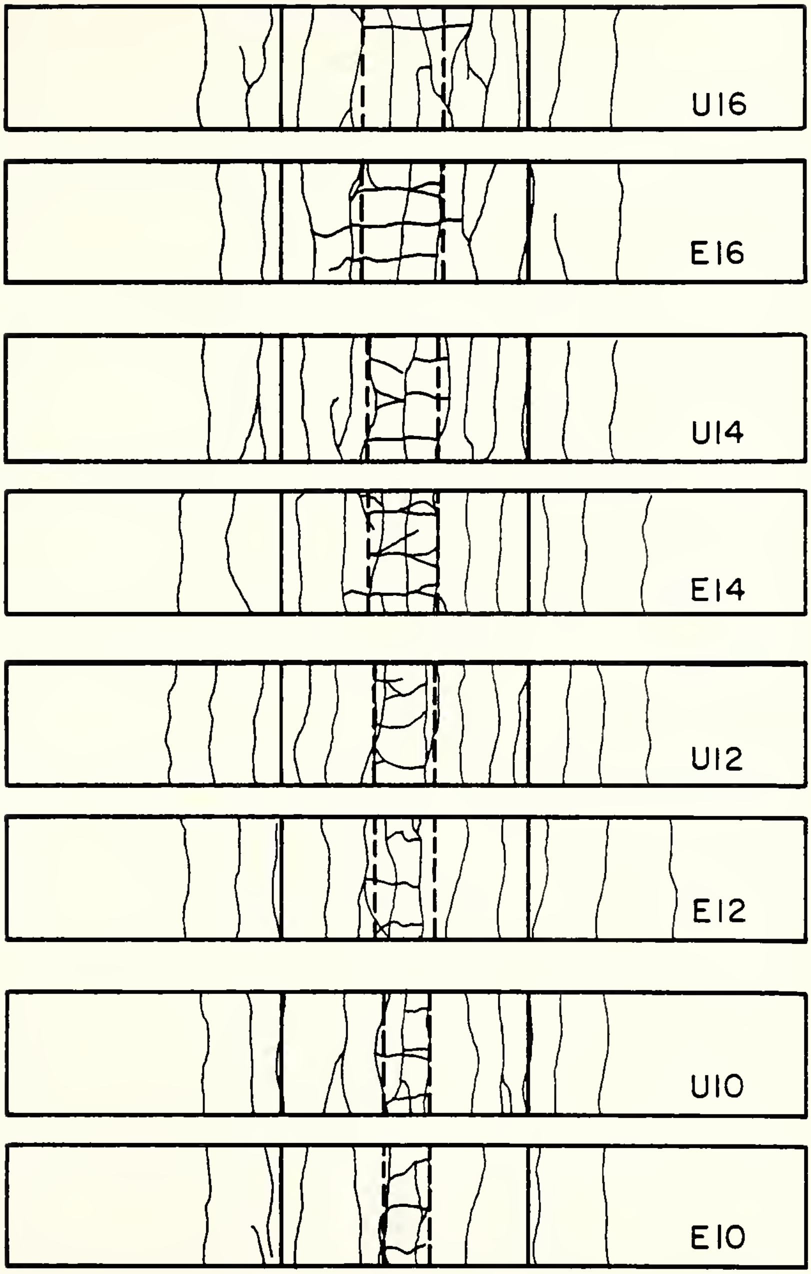

Figure B.5 Crack Patterns At Failure.

Appendix C

Table C.1. Deflections, U16.

Load South (kips)	Load North (kips)	Deflections South (inches)	Deflections North (inches)	Deflections West (inches)	Deflections East (inches)
-.01	-.00	0.003	-.010	-.001	0.002
-.00	-.00	0.005	-.012	-.001	0.002
0.24	0.25	0.009	-.011	-.001	0.002
0.49	0.50	0.014	-.009	-.001	0.002
0.59	0.61	0.019	-.010	-.001	0.002
0.80	0.81	0.023	-.011	-.001	0.002
0.99	1.00	0.027	-.011	-.001	0.002
1.20	1.23	0.034	-.010	-.001	0.003
1.38	1.41	0.042	-.009	-.001	0.003
1.47	1.50	0.047	-.008	-.001	0.004
1.57	1.60	0.054	0.056	-.001	0.005
1.76	1.80	0.068	0.070	-.001	0.008
1.98	2.01	0.105	0.115	0.003	0.013
1.99	2.03	0.115	0.118	0.004	0.014
1.99	2.03	0.112	0.118	0.004	0.014
2.00	2.04	0.117	0.119	0.004	0.014
2.00	2.04	0.117	0.120	0.004	0.014
2.00	2.04	0.115	0.120	0.004	0.014
2.00	2.04	0.119	0.120	0.004	0.014
2.00	2.03	0.119	0.120	0.005	0.014
1.99	2.03	0.119	0.121	0.004	0.014
1.99	2.02	0.120	0.121	0.005	0.014
1.99	2.02	0.119	0.122	0.005	0.014
1.99	2.02	0.120	0.123	0.005	0.014
2.01	2.04	0.129	0.130	0.005	0.015
2.01	2.05	0.130	0.132	0.005	0.015
2.17	2.21	0.139	0.142	0.006	0.017
2.36	2.40	0.165	0.175	0.008	0.020
2.28	2.35	0.186	0.211	0.011	0.023
2.28	2.33	0.189	0.211	0.010	0.023
2.28	2.33	0.190	0.212	0.011	0.027
2.31	2.36	0.197	0.216	0.011	0.027
2.32	2.36	0.201	0.219	0.011	0.027
2.32	2.35	0.203	0.219	0.012	0.027
2.49	2.53	0.221	0.237	0.012	0.027
2.55	2.59	0.231	0.248	0.013	0.029
2.96	3.01	0.299	0.314	0.019	0.036
2.96	3.00	0.303	0.316	0.019	0.037

Table C.1, continued

Load		Deflections			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
2.97	3.01	0.304	0.316	0.019	0.037
2.97	3.01	0.314	0.325	0.020	0.038
2.97	3.01	0.325	0.333	0.020	0.040
3.30	3.35	0.358	0.366	0.024	0.044
3.49	3.53	0.391	0.399	0.026	0.047
3.48	3.52	0.396	0.403	0.026	0.049
3.48	3.52	0.397	0.405	0.026	0.049
3.49	3.52	0.400	0.406	0.026	0.049
3.48	3.52	0.402	0.409	0.027	0.049
3.48	3.52	0.401	0.409	0.027	0.049
3.47	3.51	0.404	0.411	0.027	0.049
3.47	3.51	0.407	0.414	0.027	0.050
3.59	3.63	0.419	0.426	0.046	0.051
3.77	3.81	0.436	0.446	0.047	0.053
3.98	4.02	0.481	0.490	0.051	0.073
4.17	4.22	0.514	0.519	0.053	0.076
4.48	4.52	0.576	0.581	0.056	0.082
4.49	4.52	0.588	0.591	0.056	0.082
4.48	4.51	0.588	0.592	0.056	0.082
4.45	4.49	0.588	0.591	0.056	0.082
4.47	4.51	0.598	0.600	0.056	0.083
4.48	4.52	0.598	0.600	0.056	0.084
4.77	4.80	0.626	0.629	0.059	0.087
4.97	5.01	0.653	0.657	0.062	0.091
4.97	5.01	0.656	0.660	0.063	0.091
4.97	5.01	0.663	0.667	0.063	0.091
4.98	5.01	0.667	0.670	0.063	0.092
4.97	5.01	0.669	0.672	0.063	0.091
5.17	5.21	0.689	0.694	0.065	0.094
5.47	5.50	0.744	0.750	0.068	0.100
5.47	5.50	0.746	0.752	0.068	0.100
5.48	5.51	0.749	0.755	0.068	0.100
5.49	5.52	0.755	0.759	0.068	0.101
5.68	5.72	0.777	0.785	0.071	0.104
5.99	6.02	0.824	0.832	0.076	0.109
5.98	6.01	0.826	0.834	0.076	0.109
5.97	6.00	0.829	0.836	0.076	0.109
6.45	6.43	0.832	0.925	0.082	0.118

Table C.1, continued

Load South (kips)	Load North (kips)	Deflections South (inches)	Deflections North (inches)	Deflections West (inches)	Deflections East (inches)
6.44	6.44	0.831	0.927	0.082	0.118
6.43	6.45	0.919	0.930	0.082	0.118
6.43	6.46	0.921	0.930	0.082	0.118
6.43	6.46	0.922	0.932	0.082	0.118
6.44	6.47	0.924	0.933	0.082	0.118
6.46	6.49	0.929	0.937	0.082	0.118
6.48	6.51	0.933	0.940	0.083	0.118
6.49	6.51	0.939	0.945	0.083	0.118
6.48	6.51	0.955	0.960	0.083	0.121
6.47	6.50	0.143	0.142	0.083	0.121
6.61	6.64	0.156	0.157	0.084	0.123
6.98	7.00	0.207	0.209	0.089	0.128
6.97	7.00	0.212	0.217	0.089	0.128
6.99	7.01	0.220	0.227	0.090	0.130
6.98	7.00	0.223	0.228	0.090	0.130
7.17	7.20	0.248	0.253	0.092	0.133
7.42	7.45	0.288	0.293	0.096	0.137
7.43	7.46	0.290	0.294	0.096	0.138
7.49	7.52	0.303	0.308	0.097	0.139
7.50	7.52	0.313	0.315	0.098	0.140
7.50	7.52	0.315	0.317	0.098	0.140
7.50	7.52	0.320	0.321	0.098	0.140
7.50	7.52	0.326	0.327	0.098	0.142
7.51	7.53	0.331	0.329	0.098	0.142
7.50	7.51	0.334	0.332	0.098	0.142
7.51	7.52	0.337	0.335	0.098	0.142
7.50	7.52	0.342	0.339	0.098	0.142
7.51	7.52	0.347	0.344	0.098	0.143
7.61	7.63	0.356	0.354	0.099	0.145
7.82	7.84	0.382	0.383	0.101	0.146
7.96	7.99	0.411	0.417	0.103	0.151
7.97	7.99	0.416	0.423	0.103	0.152
7.98	7.99	0.429	0.444	0.103	0.154
8.24	8.26	0.501	0.576	0.109	0.169

Table C.2. Bar Strains, U16.

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.01	12	-35	0	6	0	-12
-.00	17	-35	0	6	0	-12
0.24	12	-6	9	0	20	-12
0.49	12	-3	9	6	29	-6
0.59	12	0	14	6	29	0
0.80	17	6	14	14	35	3
0.99	9	23	32	9	52	-3
1.20	14	29	32	14	58	3
1.38	20	35	43	20	64	14
1.47	35	29	52	35	64	26
1.57	67	75	156	55	72	38
1.76	107	162	263	64	96	41
1.98	313	382	561	122	113	61
1.99	321	396	570	136	119	64
1.99	330	379	564	142	107	69
2.00	327	402	570	136	119	61
2.00	336	385	567	142	107	64
2.00	339	385	567	142	107	69
2.00	327	414	576	136	124	55
2.00	321	420	576	136	130	55
1.99	327	420	582	130	127	55
1.99	333	408	587	136	119	61
1.99	330	414	582	136	124	61
1.99	339	408	587	139	119	67
2.01	365	411	602	159	110	69
2.01	365	414	608	165	113	72
2.17	376	457	660	168	133	72
2.36	446	512	744	226	162	136
2.28	451	515	750	301	414	501
2.28	443	535	764	289	431	501
2.28	443	535	758	295	437	501
2.31	460	521	761	307	437	533
2.32	451	547	776	304	469	547
2.32	475	533	773	318	457	547
2.49	504	570	828	347	506	605
2.55	527	614	836	353	538	628
2.96	660	767	1016	451	692	810
2.96	660	767	1022	454	695	810

Table C.2, continued

Load	Bar Strain					
	One	Two	Three	Four	Five	. Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
2.97	674	752	1013	466	683	819
2.97	695	764	1030	483	706	839
2.97	700	799	1053	492	732	848
3.30	773	877	1155	547	805	932
3.49	845	946	1242	608	880	1001
3.48	860	935	1239	625	874	1019
3.48	865	935	1239	631	880	1027
3.49	857	958	1253	625	891	1019
3.48	868	943	1250	637	883	1030
3.48	862	970	1259	628	897	1024
3.47	868	941	1244	645	886	1027
3.47	880	941	1256	654	883	1042
3.59	903	972	1288	671	912	1062
3.77	946	1019	1343	697	952	1114
3.98	1030	1094	1433	770	1030	1198
4.17	1094	1149	1508	825	1097	1256
4.48	1204	1276	1650	906	1227	1369
4.49	1233	1273	1647	943	1233	1383
4.48	1233	1276	1641	941	1233	1378
4.45	1233	1270	1641	938	1230	1383
4.47	1244	1279	1652	952	1244	1389
4.48	1236	1299	1664	943	1262	1386
4.77	1302	1346	1734	998	1305	1461
4.97	1363	1395	1800	1045	1366	1522
4.97	1357	1433	1817	1039	1380	1519
4.97	1366	1433	1823	1048	1398	1525
4.98	1383	1418	1817	1062	1383	1531
4.97	1375	1435	1823	1059	1407	1534
5.17	1424	1482	1890	1097	1444	1580
5.47	1534	1560	1977	1207	1548	1710
5.47	1551	1571	1977	1198	1548	1710
5.48	1551	1571	1980	1210	1548	1719
5.49	1557	1577	1991	1210	1557	1722
5.68	1600	1655	2055	1253	1624	1771
5.99	1702	1739	2159	1328	1710	1884
5.98	1713	1739	2150	1334	1705	1893
5.97	1713	1742	2165	1337	1716	1896
6.45	1855	1884	2318	1450	1858	2040

Table C.2, continued

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.44	1867	1867	2307	1461	1849	2055
6.43	1855	1893	2318	1459	1867	2049
6.43	1858	1870	2307	1467	1849	2052
6.43	1870	1870	2307	1467	1849	2061
6.44	1858	1893	2312	1459	1870	2052
6.46	1881	1884	2309	1473	1867	2066
6.48	1872	1904	2324	1470	1878	2063
6.49	1893	1896	2321	1490	1878	2081
6.48	1901	1925	2341	1499	1904	2095
6.47	1901	1925	2347	1508	1904	2098
6.61	1933	1959	2379	1531	1933	2130
6.98	2058	2043	2492	1626	2017	2246
6.97	2055	2069	2506	1624	2040	2249
6.99	2061	2087	2512	1641	2058	2257
6.98	2072	2061	2498	1655	2043	2257
7.17	2121	2136	2567	1684	2104	2309
7.42	2208	2211	2677	1745	2182	2399
7.43	2217	2197	2674	1754	2171	2405
7.49	2249	2220	2700	1774	2191	2434
7.50	2266	2228	2712	1791	2205	2443
7.50	2260	2249	2732	1783	2220	2443
7.50	2278	2237	2729	1797	2217	2451
7.50	2275	2260	2746	1794	2234	2457
7.51	2283	2266	2752	1800	2246	2460
7.50	2298	2249	2741	1815	2231	2466
7.51	2295	2272	2758	1806	2249	2457
7.50	2312	2252	2749	1823	2237	2472
7.51	2307	2278	2770	1823	2260	2474
7.61	2341	2286	2790	1841	2266	2509
7.82	2405	2341	2882	1878	2321	2576
7.96	2472	2405	3039	1910	2385	2634
7.97	2489	2414	3076	1913	2396	2648
7.98	2526	2414	3120	1930	2396	2686
8.24	2628	2567	4257	2000	2535	2796

Table C.3. Concrete Strains, U16.

Load		Concrete Strain		
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.01	-1	2	0	3
-.00	-1	1	0	3
0.24	-5	-3	-8	-6
0.49	-10	-8	-13	-13
0.59	-12	-10	-15	-15
0.80	-16	-14	-21	-21
0.99	-21	-18	-26	-26
1.20	-26	-23	-33	-34
1.38	-31	-26	-38	-39
1.47	-33	-28	-44	-43
1.57	-41	-33	-53	-51
1.76	-48	-41	-61	-62
1.98	-67	-49	-84	-87
1.99	-68	-49	-87	-89
1.99	-69	-49	-87	-90
2.00	-69	-50	-88	-89
2.00	-69	-49	-88	-90
2.00	-69	-49	-88	-90
2.00	-70	-50	-88	-90
2.00	-69	-49	-89	-91
1.99	-70	-49	-89	-90
1.99	-69	-49	-89	-91
1.99	-69	-49	-89	-91
1.99	-70	-49	-89	-92
2.01	-71	-51	-92	-97
2.01	-71	-52	-93	-98
2.17	-75	-55	-99	-104
2.36	-84	-61	-110	-116
2.28	-87	-63	-115	-117
2.28	-88	-63	-115	-117
2.28	-87	-64	-115	-117
2.31	-89	-65	-117	-122
2.32	-89	-66	-118	-121
2.32	-88	-65	-119	-122
2.49	-95	-72	-126	-132
2.55	-101	-71	-141	-140
2.96	-100	-90	-162	-173
2.96	-100	-89	-163	-173

Table C.3, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
2.97	-100	-90	-163	-174
2.97	-100	-92	-166	-178
2.97	-100	-95	-169	-182
3.30	-111	-106	-187	-202
3.49	-114	-113	-197	-216
3.48	-112	-114	-198	-217
3.48	-111	-115	-198	-219
3.49	-111	-114	-199	-218
3.48	-112	-115	-200	-219
3.48	-111	-114	-199	-219
3.47	-111	-115	-200	-221
3.47	-110	-115	-201	-222
3.59	-113	-119	-207	-229
3.77	-118	-125	-215	-238
3.98	-120	-130	-227	-254
4.17	-124	-137	-238	-268
4.48	-127	-143	-257	-289
4.49	-127	-145	-261	-294
4.48	-126	-145	-260	-294
4.45	-126	-144	-259	-294
4.47	-125	-146	-263	-295
4.48	-125	-146	-263	-297
4.77	-135	-155	-277	-314
4.97	-140	-160	-289	-325
4.97	-140	-159	-290	-325
4.97	-139	-160	-292	-328
4.98	-140	-161	-292	-329
4.97	-139	-161	-294	-330
5.17	-146	-166	-304	-340
5.47	-150	-172	-322	-358
5.47	-151	-171	-324	-360
5.48	-151	-173	-324	-360
5.49	-151	-173	-326	-363
5.68	-156	-181	-337	-376
5.99	-166	-187	-357	-393
5.98	-165	-187	-356	-394
5.97	-165	-188	-358	-395
6.45	-176	-198	-386	-422

Table C.3, continued

Load (kips)	Concrete Strain West ($\mu\epsilon$)	East ($\mu\epsilon$)	West ($\mu\epsilon$)	East ($\mu\epsilon$)
6.44	-176	-198	-386	-422
6.43	-176	-198	-386	-422
6.43	-176	-198	-386	-422
6.43	-176	-198	-386	-422
6.44	-176	-199	-388	-424
6.46	-178	-199	-390	-426
6.48	-178	-200	-391	-426
6.49	-179	-201	-391	-428
6.48	-180	-202	-396	-433
6.47	-181	-203	-398	-435
6.61	-184	-207	-404	-441
6.98	-194	-214	-424	-459
6.97	-193	-214	-425	-459
6.99	-200	-215	-427	-460
6.98	-202	-215	-427	-463
7.17	-207	-220	-438	-472
7.42	-214	-225	-452	-485
7.43	-215	-227	-452	-485
7.49	-216	-227	-457	-489
7.50	-216	-229	-457	-491
7.50	-217	-228	-458	-492
7.50	-216	-229	-458	-493
7.50	-217	-231	-458	-494
7.51	-217	-233	-460	-496
7.50	-217	-233	-460	-497
7.51	-219	-233	-461	-498
7.50	-219	-236	-463	-501
7.51	-220	-237	-467	-504
7.61	-222	-240	-471	-509
7.82	-229	-246	-482	-520
7.96	-233	-250	-488	-529
7.97	-233	-252	-488	-530
7.98	-233	-253	-488	-534
8.24	-242	-265	-503	-555

Table C.4. Deflections, U16(retest).

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
-.00	-.00	0.000	0.001	-.000	0.001
0.00	-.00	0.001	0.002	-.000	0.001
0.26	0.27	0.032	0.034	0.003	0.004
0.50	0.51	0.064	0.070	0.006	0.008
0.75	0.76	0.099	0.109	0.009	0.012
0.95	0.96	0.125	0.136	0.011	0.015
1.26	1.27	0.167	0.180	0.015	0.020
1.50	1.51	0.198	0.212	0.018	0.024
1.80	1.80	0.238	0.252	0.021	0.028
1.96	1.96	0.259	0.273	0.023	0.030
2.27	2.28	0.300	0.315	0.027	0.035
2.54	2.54	0.335	0.351	0.030	0.039
2.78	2.79	0.368	0.383	0.033	0.042
2.99	3.00	0.395	0.411	0.036	0.045
3.28	3.29	0.432	0.450	0.039	0.049
3.51	3.53	0.461	0.481	0.042	0.053
3.78	3.79	0.496	0.516	0.046	0.056
3.93	3.95	0.516	0.538	0.048	0.058
4.31	4.33	0.565	0.589	0.053	0.064
4.76	4.79	0.622	0.650	0.059	0.070
4.94	4.98	0.646	0.674	0.061	0.073
5.27	5.31	0.688	0.719	0.066	0.078
5.54	5.58	0.723	0.755	0.073	0.082
5.77	5.80	0.752	0.786	0.076	0.085
5.93	5.94	0.777	0.809	0.078	0.088
5.96	5.95	0.784	0.815	0.078	0.088
5.97	5.95	0.791	0.821	0.079	0.088
6.26	6.24	0.825	0.858	0.093	0.092
6.46	6.44	0.851	0.885	0.096	0.095
6.47	6.44	0.859	0.892	0.099	0.096
6.76	6.75	0.893	0.930	0.097	0.100
6.95	6.93	0.919	0.957	0.100	0.103
7.26	7.25	0.965	1.004	0.103	0.108
7.46	7.45	0.992	1.034	0.106	0.111
7.75	7.74	1.038	1.083	0.111	0.117
7.94	7.92	1.068	1.114	0.114	0.120
8.25	8.25	1.126	1.181	0.122	0.128
8.41	8.42	1.237	1.270	0.135	0.140

Table C.4, continued

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
8.70	8.75	1.517	1.573	0.176	0.184
8.75	8.83	1.781	1.863	0.218	0.226
8.63	8.69	1.788	1.878	0.224	0.230
7.71	7.71	1.806	1.948	0.250	0.252

Table C.5, Bar Strains.

Load (kips)	Bar Strain One ($\mu\epsilon$)	Two ($\mu\epsilon$)	Three ($\mu\epsilon$)	Four ($\mu\epsilon$)	Five ($\mu\epsilon$)	Six ($\mu\epsilon$)
-.00	0	758	5180	-3	0	9
0.00	9	767	5206	0	9	-3
0.26	64	836	5282	35	69	69
0.50	124	883	5325	78	122	145
0.75	188	964	5421	136	188	191
0.95	243	1022	5481	179	240	246
1.26	327	1094	5548	240	315	347
1.50	388	1166	5649	295	388	396
1.80	457	1242	5704	347	454	495
1.96	504	1279	5750	382	495	538
2.27	590	1369	5852	443	576	628
2.54	654	1450	5933	498	648	703
2.78	726	1534	6037	550	718	761
2.99	781	1598	6101	587	773	822
3.28	860	1667	6164	648	839	920
3.51	920	1742	6245	695	894	984
3.78	990	1835	6355	747	972	1048
3.93	1036	1878	6410	778	1010	1091
4.31	1134	2000	6541	857	1108	1201
4.76	1253	2133	6674	949	1218	1346
4.94	1305	2194	6763	984	1270	1386
5.27	1398	2301	6888	1053	1357	1479
5.54	1470	2390	6995	1114	1430	1554
5.77	1531	2451	7056	1152	1473	1647
5.93	1580	2524	7160	1192	1525	1667
5.96	1586	2535	7180	1192	1534	1676
5.97	1589	2532	7177	1195	1525	1699
6.26	1667	2634	7305	1265	1609	1765
6.46	1716	2697	7365	1297	1652	1835
6.47	1725	2715	7406	1299	1658	1844
6.76	1809	2822	7530	1369	1742	1913
6.95	1861	2900	7632	1407	1789	1971
7.26	1948	3010	7765	1476	1864	2081
7.46	2008	3123	7909	1522	1927	2127
7.75	2101	3270	8092	1583	2014	2217
7.94	2156	3421	8257	1626	2063	2278

Table C.5, continued

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
8.25	2226	3930	8639	1702	2144	2405
8.41	2301	6046	9799	1754	2202	2431
8.70	8399	8789	11307	1962	9573	8621
8.75	10092	10482	12418	1922	10795	12430
8.63	10016	10471	12447	1815	10870	13090
7.71	9319	10277	12456	1062	12048	8676

Table C.6. Concrete Strains, U16(retest).

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.00	-1	2	0	3
0.00	-1	2	0	2
0.26	-8	-4	-13	-12
0.50	-13	-11	-23	-23
0.75	-20	-17	-33	-36
0.95	-25	-22	-43	-47
1.26	-34	-29	-59	-62
1.50	-40	-35	-69	-74
1.80	-48	-43	-83	-91
1.96	-52	-47	-92	-100
2.27	-61	-54	-107	-115
2.54	-67	-62	-120	-130
2.78	-75	-68	-131	-144
2.99	-79	-74	-141	-156
3.28	-88	-81	-156	-170
3.51	-93	-87	-166	-183
3.78	-100	-95	-180	-198
3.93	-105	-98	-189	-207
4.31	-114	-108	-206	-227
4.76	-128	-120	-230	-251
4.94	-132	-124	-238	-262
5.27	-141	-133	-257	-279
5.54	-148	-140	-269	-294
5.77	-154	-145	-279	-306
5.93	-159	-150	-289	-316
5.96	-161	-152	-292	-319
5.97	-161	-154	-294	-321
6.26	-169	-162	-309	-337
6.46	-174	-166	-319	-349
6.47	-175	-168	-321	-352
6.76	-183	-176	-335	-367
6.95	-187	-181	-344	-378
7.26	-198	-190	-363	-396
7.46	-202	-197	-374	-408
7.75	-212	-207	-390	-428
7.94	-217	-212	-400	-440
8.25	-229	-222	-419	-458
8.41	-239	-229	-430	-470

Table C.6, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
8.70	-260	-250	-454	-500
8.75	-197	-275	-414	-532
8.63	-73	-279	-321	-537
7.71	156	-316	-123	-584

Table C.7. Deflections, E16.

Load		Deflections			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
0.03	0.03	0.003	0.005	-.004	-.000
0.35	0.35	0.010	0.013	-.003	-.000
0.46	0.47	0.017	0.016	-.003	0.000
0.75	0.76	0.022	0.023	-.002	0.000
0.97	0.98	0.028	0.030	-.002	-.000
1.19	1.21	0.029	0.036	-.001	0.000
1.39	1.42	0.034	0.041	-.000	0.001
1.45	1.47	0.040	0.044	-.000	0.001
1.66	1.69	0.049	0.055	0.002	0.002
1.74	1.77	0.057	0.060	0.003	0.003
1.79	1.81	0.061	0.065	0.004	0.004
1.89	1.92	0.088	0.077	0.007	0.006
1.91	1.93	0.094	0.081	0.007	0.007
1.87	1.85	0.100	0.099	0.008	0.009
1.90	1.92	0.108	0.136	0.010	0.010
1.89	1.92	0.118	0.143	0.011	0.011
1.93	1.96	0.133	0.156	0.014	0.013
2.24	2.27	0.156	0.178	0.017	0.016
2.19	2.21	0.169	0.207	0.021	0.020
2.17	2.24	0.206	0.225	0.023	0.021
2.21	2.25	0.208	0.223	0.023	0.021
2.23	2.26	0.213	0.224	0.024	0.021
2.45	2.48	0.234	0.241	0.025	0.023
2.47	2.50	0.247	0.248	0.027	0.024
2.62	2.65	0.262	0.263	0.029	0.026
2.62	2.50	0.266	0.287	0.028	0.028
2.41	2.52	0.298	0.305	0.030	0.026
2.72	2.76	0.322	0.322	0.031	0.029
2.93	2.97	0.351	0.349	0.035	0.032
2.96	3.00	0.374	0.373	0.037	0.035
3.20	3.24	0.396	0.393	0.040	0.037
3.44	3.49	0.429	0.425	0.044	0.041
3.49	3.53	0.443	0.441	0.045	0.043
3.71	3.75	0.468	0.464	0.048	0.046
3.93	3.97	0.498	0.497	0.052	0.049
3.95	3.98	0.512	0.513	0.053	0.051
3.94	3.98	0.513	0.516	0.053	0.051
4.25	4.30	0.546	0.553	0.057	0.055

Table C.7, continued

Load		Deflections			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
4.41	4.47	0.590	0.586	0.061	0.058
4.45	4.49	0.616	0.616	0.063	0.060
4.70	4.75	0.643	0.648	0.066	0.063
4.91	4.96	0.673	0.682	0.069	0.067
4.92	4.97	0.690	0.712	0.071	0.068
5.23	5.27	0.722	0.747	0.075	0.072
5.43	5.49	0.755	0.784	0.079	0.076
5.73	5.78	0.813	0.842	0.085	0.081
5.90	5.96	0.858	0.871	0.089	0.084
6.21	6.27	0.909	0.921	0.095	0.089
6.41	6.47	0.941	0.955	0.099	0.093
6.43	6.49	0.958	0.971	0.101	0.094
6.44	6.50	0.962	0.975	0.101	0.095
6.40	6.45	0.965	0.978	0.101	0.095
6.45	6.51	0.976	0.990	0.103	0.096
6.66	6.71	0.997	1.011	0.105	0.098
6.79	6.84	1.014	1.028	0.107	0.100
6.90	6.95	1.033	1.046	0.109	0.102
6.91	6.97	1.051	1.065	0.112	0.104
7.21	7.27	1.089	1.103	0.117	0.108
7.41	7.47	1.121	1.136	0.121	0.112
7.42	7.48	1.125	1.140	0.121	0.113
7.45	7.51	1.170	1.187	0.129	0.119
7.65	7.71	1.203	1.222	0.136	0.124
7.66	7.73	1.240	1.264	0.146	0.133
7.47	7.55	1.263	1.291	0.154	0.141
7.34	7.42	1.273	1.303	0.158	0.145

Table C.8. Bar Strains, E16.

Load (kips)	Bar Strain One ($\mu\epsilon$)	Two ($\mu\epsilon$)	Three ($\mu\epsilon$)	Four ($\mu\epsilon$)	Five ($\mu\epsilon$)	Six ($\mu\epsilon$)
0.03	-6	0	0	-6	3	9
0.35	6	14	14	9	9	14
0.46	6	14	14	3	9	14
0.75	6	29	9	20	23	-3
0.97	17	29	29	14	14	20
1.19	23	32	23	29	26	9
1.39	26	32	29	32	23	29
1.45	29	35	26	35	29	14
1.66	32	46	46	61	41	55
1.74	52	58	46	81	43	67
1.79	101	101	41	90	64	46
1.89	205	174	58	104	78	87
1.91	217	191	69	113	84	87
1.87	234	197	78	116	87	93
1.90	258	226	78	136	127	72
1.89	344	327	240	142	119	116
1.93	454	454	356	153	145	133
2.24	556	573	466	197	188	179
2.19	596	622	512	330	301	272
2.17	645	686	550	396	356	292
2.21	648	677	570	394	350	318
2.23	651	680	570	396	350	315
2.45	721	752	622	423	379	341
2.47	764	805	666	446	399	362
2.62	819	860	718	475	425	385
2.62	819	877	712	486	440	365
2.41	816	854	724	504	463	411
2.72	868	920	752	533	495	402
2.93	943	1001	842	616	570	492
2.96	1001	1074	900	680	640	547
3.20	1068	1137	946	724	674	559
3.44	1161	1262	1048	805	755	616
3.49	1192	1294	1097	848	787	683
3.71	1265	1380	1149	906	845	712
3.93	1349	1467	1247	981	909	805
3.95	1366	1502	1282	1007	943	839
3.94	1375	1511	1276	1013	946	810
4.25	1479	1618	1378	1091	1010	900

Table C.8, continued

Load			Bar Strain			
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
4.41	1557	1702	1433	1158	1074	943
4.45	1577	1745	1493	1178	1111	1039
4.70	1650	1823	1551	1233	1163	1088
4.91	1713	1913	1615	1294	1221	1129
4.92	1725	1933	1658	1334	1239	1198
5.23	1815	2043	1728	1392	1308	1230
5.43	1887	2121	1803	1470	1378	1305
5.73	1994	2231	1925	1560	1459	1430
5.90	2063	2304	1968	1624	1511	1459
6.21	2171	2408	2084	1705	1600	1592
6.41	2246	2480	2153	1771	1655	1658
6.43	2278	2509	2185	1797	1679	1693
6.44	2281	2512	2176	1803	1699	1676
6.40	2269	2515	2191	1800	1696	1713
6.45	2298	2538	2194	1823	1713	1702
6.66	2344	2593	2260	1867	1742	1762
6.79	2396	2648	2286	1898	1783	1765
6.90	2437	2689	2327	1933	1820	1800
6.91	2451	2712	2344	1959	1844	1838
7.21	2541	2804	2431	2026	1910	1901
7.41	2610	2880	2495	2087	1974	1965
7.42	2605	2880	2509	2089	1971	2000
7.45	2622	3351	2532	2058	2092	2069
7.65	2645	3788	2599	1997	2217	2162
7.66	2535	5131	2665	1809	2367	2364
7.47	2382	6280	2547	1618	2460	2396

Table C.9. Concrete Strains, E16.

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
0.03	-3	1	1	1
0.35	-9	-5	-10	-7
0.46	-11	-8	-13	-11
0.75	-17	-13	-23	-18
0.97	-21	-18	-29	-25
1.19	-27	-23	-38	-32
1.39	-32	-28	-43	-38
1.45	-33	-29	-47	-41
1.66	-49	-34	-70	-54
1.74	-54	-37	-76	-59
1.79	-57	-38	-82	-63
1.89	-69	-43	-101	-73
1.91	-72	-45	-106	-75
1.87	-68	-49	-115	-80
1.90	-68	-51	-121	-85
1.89	-71	-53	-131	-86
1.93	-75	-56	-138	-91
2.24	-84	-67	-154	-106
2.19	-85	-68	-156	-107
2.17	-86	-70	-160	-110
2.21	-87	-71	-161	-111
2.23	-89	-72	-162	-113
2.45	-95	-77	-175	-122
2.47	-95	-78	-178	-125
2.62	-100	-83	-188	-131
2.62	-97	-82	-186	-130
2.41	-94	-81	-183	-130
2.72	-100	-88	-197	-140
2.93	-106	-93	-211	-148
2.96	-105	-92	-220	-156
3.20	-110	-108	-233	-166
3.44	-115	-110	-250	-174
3.49	-117	-110	-257	-176
3.71	-121	-120	-270	-187
3.93	-127	-118	-286	-195
3.95	-126	-123	-289	-197
3.94	-128	-125	-293	-198
4.25	-136	-132	-310	-210

Table C.9, continued

Load (kips)	Concrete Strain West ($\mu\epsilon$)	East ($\mu\epsilon$)	West ($\mu\epsilon$)	East ($\mu\epsilon$)
4.41	-138	-137	-323	-217
4.45	-141	-140	-331	-221
4.70	-146	-145	-346	-231
4.91	-148	-150	-358	-240
4.92	-147	-151	-362	-243
5.23	-153	-156	-379	-254
5.43	-161	-161	-392	-265
5.73	-169	-170	-415	-283
5.90	-173	-173	-424	-290
6.21	-181	-181	-443	-307
6.41	-187	-187	-456	-315
6.43	-185	-188	-458	-321
6.44	-187	-190	-460	-324
6.40	-187	-190	-459	-325
6.45	-191	-194	-465	-330
6.66	-196	-197	-474	-337
6.79	-198	-201	-481	-342
6.90	-202	-203	-488	-348
6.91	-205	-207	-492	-355
7.21	-215	-217	-509	-370
7.41	-222	-225	-519	-382
7.42	-222	-226	-519	-383
7.45	-232	-250	-503	-417
7.65	-233	-264	-469	-441
7.66	-249	-283	-397	-472

Table C.10. Deflections, U14.

Load		Deflections			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
0.00	-.02	-.000	-.000	0.000	0.000
-.00	-.01	0.001	-.000	0.000	0.000
0.27	0.25	0.007	0.006	0.000	0.000
0.51	0.48	0.012	0.010	0.000	0.001
0.77	0.75	0.019	0.017	0.001	0.001
0.99	0.97	0.023	0.022	0.001	0.002
1.27	1.25	0.031	0.028	0.001	0.002
1.48	1.46	0.037	0.034	0.002	0.002
1.50	1.47	0.039	0.036	0.002	0.003
1.76	1.73	0.047	0.044	0.002	0.004
1.98	1.95	0.059	0.061	0.004	0.005
2.19	2.15	0.091	0.109	0.008	0.013
2.26	2.23	0.102	0.120	0.009	0.015
2.37	2.33	0.120	0.134	0.012	0.018
2.47	2.44	0.161	0.155	0.015	0.021
2.51	2.47	0.180	0.168	0.016	0.022
2.53	2.48	0.184	0.173	0.016	0.023
2.77	2.73	0.214	0.202	0.018	0.026
2.99	2.94	0.267	0.242	0.022	0.030
3.04	2.98	0.289	0.272	0.024	0.033
3.15	3.11	0.327	0.304	0.029	0.038
3.25	3.17	0.342	0.337	0.030	0.040
3.51	3.44	0.377	0.372	0.033	0.044
3.77	3.71	0.416	0.408	0.037	0.049
4.00	3.93	0.452	0.443	0.040	0.054
4.02	3.94	0.478	0.462	0.042	0.056
4.28	4.21	0.503	0.487	0.045	0.059
4.51	4.45	0.550	0.522	0.049	0.064
4.52	4.43	0.560	0.529	0.049	0.065
4.42	4.29	0.564	0.543	0.051	0.067
4.81	4.73	0.612	0.609	0.055	0.073
5.00	4.92	0.644	0.639	0.058	0.077
5.02	4.91	0.668	0.656	0.059	0.079
5.28	5.20	0.695	0.686	0.062	0.082
5.51	5.41	0.733	0.722	0.065	0.086
5.79	5.70	0.794	0.772	0.070	0.093
6.00	5.91	0.827	0.802	0.073	0.096
5.92	5.89	0.846	0.807	0.074	0.097

Table C.10, continued

Load		Deflections			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
6.31	6.23	0.917	0.862	0.078	0.103
6.50	6.40	0.946	0.891	0.081	0.107
6.50	6.38	0.955	0.897	0.082	0.108
6.81	6.72	1.005	0.952	0.087	0.113
7.00	6.90	1.034	0.981	0.089	0.117
7.30	7.21	1.094	1.042	0.095	0.124
7.51	7.40	1.130	1.078	0.099	0.128
7.77	7.67	1.195	1.143	0.105	0.136
7.98	7.87	1.247	1.211	0.112	0.146
8.17	8.06	1.351	1.429	0.131	0.175
8.26	8.18	1.422	1.563	0.142	0.190
8.28	8.19	1.466	1.649	0.149	0.200
8.38	8.33	1.575	1.852	0.168	0.225
8.42	8.36	1.619	1.921	0.175	0.234
8.47	8.39	1.819	-.102	0.204	0.268
8.46	8.38	0.272	-.102	0.207	0.271
8.46	8.37	0.286	-.102	0.208	0.273
8.71	8.68	0.492	-.102	0.237	0.306
4.17	4.15	0.778	-.102	0.414	0.470

Table C.11. Bar Strains, U14.

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
0.00	29	12	-69	-43	29	-61
-.00	23	-41	14	-55	-29	9
0.27	26	-35	20	-52	-26	14
0.51	41	23	-58	-38	41	-55
0.77	41	23	-58	-26	41	-49
0.99	38	-26	32	-41	-17	26
1.27	43	-14	43	-35	-9	32
1.48	49	-14	43	-29	-6	32
1.50	49	-14	43	-29	-6	41
1.76	64	43	-32	-9	75	-26
1.98	61	3	55	-6	49	81
2.19	81	20	69	327	399	260
2.26	101	72	-9	373	492	214
2.37	119	93	35	411	530	237
2.47	139	116	78	454	570	269
2.51	136	69	174	478	521	365
2.53	136	69	179	483	527	370
2.77	148	81	220	547	602	425
2.99	191	159	205	645	767	428
3.04	208	150	333	686	735	553
3.15	628	544	359	729	845	512
3.25	761	671	414	741	865	527
3.51	860	770	457	822	949	605
3.77	952	851	518	912	1036	686
4.00	1059	955	590	996	1117	776
4.02	1094	949	718	1033	1088	900
4.28	1149	1004	758	1088	1149	949
4.51	1253	1123	894	1169	1230	1030
4.52	1276	1163	943	1187	1242	1048
4.42	1273	1210	865	1192	1349	1051
4.81	1357	1256	1036	1279	1369	1224
5.00	1418	1323	1108	1323	1421	1262
5.02	1453	1401	1074	1343	1516	1227
5.28	1505	1415	1207	1407	1496	1343
5.51	1592	1540	1204	1464	1641	1337
5.79	1684	1638	1299	1551	1731	1418
6.00	1736	1652	1444	1615	1713	1548
5.92	1745	1705	1369	1615	1794	1479

Table C.11, continued

Load			Bar Strain			
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.31	1841	1762	1554	1713	1815	1647
6.50	1910	1870	1528	1768	1948	1621
6.50	1922	1878	1557	1777	1956	1632
6.81	2000	1916	1725	1861	1980	1789
7.00	2066	1971	1786	1910	2037	1838
7.30	2176	2075	1896	2014	2150	1933
7.51	2266	2194	1875	2095	2298	1919
7.77	2570	2298	1985	2199	2431	1991
7.98	5261	2419	2084	2266	2550	1994
8.17	7044	2495	2217	2315	2567	2049
8.26	7600	2460	2362	2362	2509	2156
8.28	7883	2500	2315	2402	2634	2095
8.38	5970	2512	2376	2524	3548	2127
8.42	5096	2529	2399	2668	4460	2139
8.47	2868	6630	2509	6407	6836	2243
8.46	2764	7649	2506	6633	7021	2249
8.46	2718	8118	2445	6801	7206	2179
8.71	2457	10407	2573	8847	9559	2292
4.17	1815	10803	831	8566	17020	527

Table C.12. Concrete Strains, U14.

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
0.00	-1	0	0	2
-.00	-1	0	0	2
0.27	-5	-5	-7	-6
0.51	-9	-10	-12	-12
0.77	-13	-15	-18	-19
0.99	-18	-19	-24	-26
1.27	-23	-27	-32	-36
1.48	-28	-32	-38	-43
1.50	-29	-34	-41	-46
1.76	-33	-42	-48	-55
1.98	-42	-46	-62	-66
2.19	-47	-64	-69	-89
2.26	-49	-69	-74	-98
2.37	-57	-73	-84	-110
2.47	-64	-79	-95	-119
2.51	-68	-82	-99	-122
2.53	-70	-84	-102	-125
2.77	-78	-94	-115	-139
2.99	-88	-105	-132	-160
3.04	-92	-113	-144	-174
3.15	-95	-119	-149	-187
3.25	-99	-120	-153	-190
3.51	-100	-125	-166	-207
3.77	-107	-135	-179	-226
4.00	-111	-141	-190	-240
4.02	-115	-144	-197	-248
4.28	-121	-152	-209	-261
4.51	-126	-156	-217	-274
4.52	-126	-157	-219	-275
4.42	-125	-155	-216	-273
4.81	-135	-167	-235	-296
5.00	-140	-166	-243	-304
5.02	-141	-165	-246	-309
5.28	-146	-171	-256	-323
5.51	-151	-174	-265	-334
5.79	-158	-180	-280	-355
6.00	-163	-182	-289	-365
5.92	-161	-181	-288	-365

Table C.12, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.31	-170	-192	-305	-386
6.50	-174	-196	-313	-395
6.50	-174	-195	-313	-396
6.81	-184	-207	-331	-417
7.00	-190	-211	-339	-427
7.30	-199	-222	-358	-448
7.51	-202	-227	-366	-460
7.77	-211	-236	-384	-481
7.98	-217	-243	-396	-493
8.17	-222	-253	-408	-510
8.26	-224	-258	-413	-517
8.28	-224	-259	-416	-517
8.38	-227	-263	-422	-522
8.42	-229	-263	-427	-524
8.47	-237	-268	-439	-513
8.46	-237	-269	-440	-513
8.46	-238	-270	-441	-514
8.71	-245	-285	-456	-524
4.17	-154	-27	-220	-318

Table C.13. Deflections, E14.

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
-.01	-.02	-.000	-.000	0.000	-.000
-.01	-.01	-.000	0.000	-.000	-.000
0.26	0.24	0.006	0.005	0.001	-.000
0.51	0.48	0.011	0.009	0.001	0.000
0.76	0.75	0.018	0.016	0.001	0.001
1.00	0.99	0.024	0.022	0.002	0.002
1.27	1.25	0.031	0.029	0.002	0.002
1.48	1.47	0.039	0.036	0.003	0.002
1.76	1.75	0.049	0.047	0.004	0.004
1.86	1.85	0.075	0.067	0.007	0.007
1.97	1.96	0.100	0.098	0.011	0.010
2.21	2.20	0.154	0.153	0.016	0.015
2.23	2.22	0.171	0.178	0.020	0.020
2.49	2.48	0.205	0.213	0.025	0.024
2.76	2.76	0.254	0.278	0.029	0.028
2.85	2.95	0.307	0.308	0.033	0.032
2.95	2.93	0.325	0.311	0.033	0.033
2.94	2.93	0.329	0.316	0.034	0.033
3.39	3.39	0.384	0.370	0.041	0.040
3.50	3.49	0.401	0.386	0.043	0.041
3.51	3.49	0.414	0.398	0.044	0.042
3.77	3.77	0.444	0.428	0.048	0.046
3.96	3.94	0.480	0.463	0.053	0.051
4.26	4.25	0.537	0.540	0.062	0.060
4.49	4.48	0.569	0.577	0.065	0.064
4.77	4.75	0.615	0.625	0.070	0.069
4.98	4.97	0.645	0.656	0.074	0.072
4.95	4.91	0.676	0.665	0.075	0.074
5.01	4.97	0.691	0.677	0.076	0.075
5.26	5.25	0.718	0.705	0.079	0.078
5.49	5.48	0.754	0.739	0.084	0.082
5.82	5.81	0.811	0.820	0.091	0.089
6.00	5.98	0.837	0.845	0.093	0.092
6.01	5.97	0.852	0.858	0.095	0.094
6.27	6.25	0.889	0.896	0.100	0.098
6.49	6.46	0.921	0.926	0.104	0.102
6.49	6.46	0.924	0.929	0.104	0.102
6.81	6.80	0.982	0.985	0.111	0.109

Table C.13, continued

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
6.99	6.96	1.013	1.012	0.115	0.113
7.15	7.14	1.066	1.063	0.124	0.121
7.33	7.31	1.088	1.087	0.127	0.124
7.49	7.46	1.119	1.116	0.132	0.128
7.49	7.47	1.128	1.125	0.133	0.130
7.26	7.24	1.193	1.193	0.155	0.149
7.20	7.17	1.198	1.197	0.157	0.151
6.96	6.94	1.211	1.215	0.165	0.159

Table C.14. Bar Strains, E14.

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.01	6	38	-81	-3	20	-67
-.01	-3	-14	6	-12	-43	9
0.26	9	-14	12	-12	-38	12
0.51	17	43	-69	9	26	-61
0.76	14	-3	17	0	-32	23
1.00	17	9	23	0	-32	29
1.27	32	61	-58	20	43	-41
1.48	35	17	32	17	-14	41
1.76	49	75	-41	41	64	-29
1.86	324	275	75	58	81	-12
1.97	564	466	284	69	93	6
2.21	660	556	370	90	130	38
2.23	686	564	382	260	350	159
2.49	781	642	449	359	449	246
2.76	883	683	616	411	457	388
2.85	946	738	677	457	506	437
2.95	967	802	614	486	587	379
2.94	958	767	709	478	535	457
3.39	1091	888	825	567	628	553
3.50	1137	972	778	614	724	515
3.51	1152	946	894	622	689	622
3.77	1227	1001	955	671	732	671
3.96	1325	1120	952	744	857	648
4.26	1409	1152	1120	952	1062	1140
4.49	1482	1215	1192	1007	1111	1244
4.77	1571	1352	1195	1085	1236	1273
4.98	1635	1360	1340	1117	1227	1421
4.95	1638	1418	1276	1146	1302	1357
5.01	1658	1444	1305	1163	1323	1378
5.26	1722	1441	1435	1187	1299	1514
5.49	1789	1502	1511	1236	1369	1580
5.82	1887	1595	1603	1320	1459	1684
6.00	1936	1647	1655	1363	1502	1734
6.01	1956	1713	1603	1404	1586	1687
6.27	2029	1780	1661	1447	1658	1762
6.49	2089	1835	1722	1493	1719	1829
6.49	2092	1835	1722	1499	1725	1829
6.81	2185	1881	1901	1566	1754	1997

Table C.14, continued

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.99	2217	1936	1956	1606	1800	2052
7.15	2223	2014	2043	1609	1925	2144
7.33	2275	2098	2008	1658	2037	2116
7.49	2307	2147	2063	1679	2110	2182
7.49	2295	2113	2153	1655	2078	2260
7.26	2144	2315	2144	1554	2376	2309
7.20	2133	2385	2052	1557	2451	2226
6.96	2032	2399	2089	1490	2437	2260

Table C.15. Concrete Strains, E14.

Load (kips)	Concrete Strain West ($\mu\epsilon$)	East ($\mu\epsilon$)	West ($\mu\epsilon$)	East ($\mu\epsilon$)
-.01	3	2	3	0
-.01	3	2	3	0
0.26	-3	-3	-6	-6
0.51	-8	-7	-13	-13
0.76	-15	-12	-20	-18
1.00	-21	-17	-27	-23
1.27	-27	-21	-35	-29
1.48	-32	-27	-43	-37
1.76	-43	-33	-57	-45
1.86	-54	-38	-72	-50
1.97	-60	-45	-81	-59
2.21	-69	-60	-104	-78
2.23	-71	-74	-107	-95
2.49	-89	-70	-130	-106
2.76	-104	-76	-155	-121
2.85	-110	-80	-168	-132
2.95	-110	-80	-170	-135
2.94	-106	-82	-169	-137
3.39	-120	-93	-192	-164
3.50	-122	-94	-197	-170
3.51	-123	-91	-200	-170
3.77	-132	-98	-215	-180
3.96	-146	-100	-237	-181
4.26	-156	-112	-257	-197
4.49	-161	-117	-266	-205
4.77	-168	-125	-282	-218
4.98	-174	-129	-291	-227
4.95	-172	-129	-293	-227
5.01	-175	-130	-297	-233
5.26	-183	-137	-309	-243
5.49	-190	-141	-322	-251
5.82	-203	-151	-343	-268
6.00	-206	-153	-350	-273
6.01	-204	-153	-351	-276
6.27	-217	-164	-371	-290
6.49	-223	-169	-382	-299
6.49	-223	-169	-382	-299
6.81	-235	-180	-404	-318

Table C.15, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.99	-241	-185	-411	-328
7.15	-217	-207	-399	-359
7.33	-222	-212	-407	-367
7.49	-222	-220	-411	-378
7.49	-218	-225	-414	-385
7.26	-84	-154	-332	-325
7.20	-50	-141	-328	-309
6.96	0	-120	-318	-280

Table C.16. Deflections, U12.

Load South (kips)	Load North (kips)	Deflections South (inches)	Deflections North (inches)	Deflections West (inches)	Deflections East (inches)
0.00	0.00	-.000	0.003	0.001	-.001
0.27	0.27	0.009	0.006	0.000	0.001
0.50	0.50	0.017	0.010	-.000	0.002
0.78	0.78	0.026	0.016	-.001	0.003
0.99	0.99	0.036	0.019	-.002	0.005
0.87	0.88	0.040	0.017	-.004	0.006
0.99	0.99	0.043	0.019	-.003	0.006
1.26	1.27	0.052	0.025	-.003	0.007
1.48	1.48	0.062	0.030	-.003	0.008
1.76	1.76	0.078	0.039	-.004	0.010
1.92	1.91	0.100	0.066	-.003	0.014
1.91	1.89	0.101	0.071	-.003	0.014
2.24	2.24	0.183	0.121	0.003	0.021
2.49	2.49	0.239	0.154	0.008	0.026
2.51	2.50	0.279	0.166	0.009	0.028
2.76	2.75	0.300	0.186	0.011	0.030
2.67	2.64	0.308	0.215	0.013	0.033
2.67	2.65	0.309	0.216	0.013	0.033
2.99	2.97	0.342	0.279	0.017	0.036
3.01	2.99	0.358	0.296	0.019	0.037
3.26	3.23	0.384	0.328	0.021	0.039
3.49	3.47	0.423	0.386	0.026	0.042
3.46	3.43	0.431	0.391	0.026	0.042
3.54	3.52	0.451	0.410	0.028	0.044
3.77	3.74	0.473	0.434	0.031	0.045
3.98	3.96	0.510	0.468	0.035	0.048
4.27	4.24	0.578	0.522	0.042	0.052
4.48	4.46	0.614	0.577	0.047	0.055
4.48	4.44	0.640	0.606	0.050	0.056
4.75	4.72	0.666	0.635	0.053	0.058
4.97	4.93	0.717	0.680	0.062	0.064
4.95	4.90	0.724	0.687	0.063	0.064
4.97	4.92	0.732	0.695	0.064	0.064
5.32	5.28	0.776	0.743	0.070	0.069
5.50	5.45	0.809	0.777	0.074	0.072
5.52	5.45	0.837	0.805	0.077	0.074
5.76	5.71	0.866	0.834	0.080	0.076
5.97	5.92	0.900	0.868	0.084	0.080

Table C.16, continued

Load		Deflections			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
5.98	5.92	0.938	0.914	0.094	0.084
6.32	6.27	0.987	0.968	0.101	0.091
6.47	6.41	1.028	1.014	0.109	0.099
6.39	6.32	1.081	1.067	0.120	0.112
5.79	5.73	1.116	1.118	0.138	0.135
1.49	1.44	1.251	1.343	0.300	0.286

Table C.17. Bar Strains, U12.

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
0.00	-14	-35	32	-17	-35	26
0.27	-9	6	-26	-9	6	-32
0.50	3	17	-14	3	12	-26
0.78	-6	-20	49	-6	-26	35
0.99	6	-14	55	6	-20	41
0.87	6	-14	55	6	-26	41
0.99	14	23	-3	14	23	-17
1.26	12	-9	64	12	-14	46
1.48	26	38	17	29	32	-6
1.76	43	49	23	58	43	6
1.92	81	87	67	156	72	32
1.91	72	46	124	148	29	90
2.24	587	544	475	339	98	72
2.49	747	697	628	504	133	104
2.51	796	741	677	541	150	127
2.76	862	807	738	596	159	142
2.67	862	807	738	625	411	524
2.67	860	773	802	616	370	590
2.99	952	860	883	671	492	741
3.01	993	935	854	706	573	715
3.26	1068	964	978	747	573	828
3.49	1161	1059	1071	831	640	909
3.46	1166	1068	1077	842	651	926
3.54	1210	1108	1108	874	683	955
3.77	1279	1210	1117	932	764	941
3.98	1346	1291	1184	1004	819	1013
4.27	1476	1424	1288	1097	900	1106
4.48	1531	1485	1360	1169	961	1178
4.48	1543	1511	1409	1207	1010	1215
4.75	1589	1543	1525	1253	1004	1328
4.97	1664	1661	1583	1337	1100	1331
4.95	1658	1661	1589	1349	1106	1337
4.97	1664	1670	1600	1366	1120	1352
5.32	1742	1702	1731	1456	1140	1493
5.50	1797	1751	1780	1528	1189	1554
5.52	1820	1771	1800	1577	1233	1580
5.76	1890	1864	1800	1647	1314	1577
5.97	1951	1925	1858	1722	1357	1626

Table C.17, continued

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
5.98	1977	1933	1875	1823	1395	1644
6.32	2061	1977	2000	1945	1421	1768
6.47	2121	2003	2017	2104	1470	1777
6.39	2142	2029	1907	2350	1560	1667
5.79	2011	1861	1786	2385	1459	1580
1.49	909	964	857	587	706	706

Table C.18. Concrete Strains, U12.

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
0.00	2	0	1	-1
0.27	-4	-6	-7	-7
0.50	-10	-11	-13	-13
0.78	-18	-18	-23	-22
0.99	-24	-24	-30	-29
0.87	-27	-25	-31	-31
0.99	-28	-26	-34	-33
1.26	-35	-32	-41	-39
1.48	-40	-38	-49	-47
1.76	-49	-46	-59	-59
1.92	-59	-59	-74	-76
1.91	-59	-59	-74	-77
2.24	-72	-74	-93	-100
2.49	-84	-89	-114	-124
2.51	-89	-95	-120	-130
2.76	-95	-101	-132	-141
2.67	-95	-100	-131	-139
2.67	-95	-101	-131	-141
2.99	-106	-112	-147	-156
3.01	-91	-115	-152	-160
3.26	-99	-124	-164	-171
3.49	-105	-130	-176	-183
3.46	-106	-131	-177	-184
3.54	-110	-137	-184	-192
3.77	-115	-144	-192	-202
3.98	-119	-152	-202	-212
4.27	-126	-165	-216	-230
4.48	-130	-173	-228	-242
4.48	-135	-177	-237	-248
4.75	-142	-186	-248	-258
4.97	-144	-185	-273	-255
4.95	-142	-187	-274	-256
4.97	-141	-187	-276	-256
5.32	-150	-199	-291	-274
5.50	-155	-206	-300	-284
5.52	-157	-213	-309	-295
5.76	-163	-222	-318	-307
5.97	-167	-228	-326	-319

Table C.18, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
5.98	-172	-181	-339	-305
6.32	-186	-144	-360	-291
6.47	-197	-130	-376	-215
6.39	-212	-195	-396	-86
5.79	-188	-193	-377	-42
1.49	-5	-366	-159	-328

Table C.19. Deflections, E12.

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
-.01	-.00	-.001	0.000	0.000	0.000
0.26	0.25	0.006	0.006	0.000	0.001
0.50	0.49	0.013	0.012	0.000	0.001
0.77	0.76	0.021	0.018	0.000	0.002
0.98	0.98	0.027	0.024	0.000	0.003
1.26	1.25	0.038	0.032	0.000	0.004
1.47	1.46	0.049	0.051	0.001	0.006
1.50	1.49	0.054	0.060	0.001	0.006
1.49	1.47	0.054	0.061	0.001	0.006
1.79	1.78	0.068	0.080	0.002	0.008
1.77	1.74	0.078	0.097	0.002	0.009
1.88	1.87	0.112	0.123	0.002	0.010
1.99	1.98	0.122	0.134	0.004	0.012
2.10	2.09	0.184	0.188	0.013	0.023
2.14	2.13	0.224	0.206	0.016	0.028
2.21	2.19	0.233	0.211	0.017	0.028
2.49	2.48	0.260	0.237	0.019	0.032
-.01	-.00	-.001	0.000	0.000	0.000
0.26	0.25	0.006	0.006	0.000	0.001
0.50	0.49	0.013	0.012	0.000	0.001
0.77	0.76	0.021	0.018	0.000	0.002
0.98	0.98	0.027	0.024	0.000	0.003
1.26	1.25	0.038	0.032	0.000	0.004
1.47	1.46	0.049	0.051	0.001	0.006
1.50	1.49	0.054	0.060	0.001	0.006
1.49	1.47	0.054	0.061	0.001	0.006
1.79	1.78	0.068	0.080	0.002	0.008
1.77	1.74	0.078	0.097	0.002	0.009
1.88	1.87	0.112	0.123	0.002	0.010
1.99	1.98	0.122	0.134	0.004	0.012
2.10	2.09	0.184	0.188	0.013	0.023
2.14	2.13	0.224	0.206	0.016	0.028
2.21	2.19	0.233	0.211	0.017	0.028
2.49	2.48	0.260	0.237	0.019	0.032
3.99	3.95	0.553	0.511	0.043	0.067
4.04	3.99	0.569	0.524	0.044	0.069
4.27	4.23	0.596	0.550	0.047	0.073
4.50	4.47	0.632	0.588	0.051	0.079

Table C.19, continued

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
4.53	4.48	0.656	0.625	0.054	0.082
4.76	4.73	0.693	0.662	0.057	0.087
4.91	4.88	0.728	0.686	0.059	0.091
4.94	4.90	0.734	0.690	0.060	0.091
5.00	4.95	0.764	0.713	0.062	0.094
5.27	5.23	0.805	0.754	0.066	0.100
5.50	5.46	0.842	0.792	0.070	0.105
5.76	5.72	0.903	0.848	0.076	0.114
5.94	5.89	0.970	0.883	0.079	0.119
5.93	5.88	0.993	0.900	0.082	0.123
6.02	5.98	1.035	0.941	0.088	0.132
6.23	6.20	1.089	0.993	0.097	0.143
6.03	6.01	1.104	1.012	0.102	0.151

Table C.20. Bar Strains, E12.

Load (kips)	Bar Strain One ($\mu\epsilon$)	Two ($\mu\epsilon$)	Three ($\mu\epsilon$)	Four ($\mu\epsilon$)	Five ($\mu\epsilon$)	Six ($\mu\epsilon$)
-.01	-23	23	-43	-26	32	-23
0.26	-17	26	-43	-20	32	-17
0.50	-17	-9	29	-23	-3	35
0.77	-12	-9	29	-12	3	46
0.98	-3	38	-26	-3	49	-6
1.26	3	49	-23	3	55	0
1.47	9	55	-17	20	67	17
1.50	6	14	52	17	35	75
1.49	6	20	52	17	41	75
1.79	20	67	-6	41	87	38
1.77	26	72	-12	41	87	43
1.88	29	41	69	43	64	107
1.99	35	46	75	55	75	130
2.10	266	593	533	541	561	292
2.14	321	642	637	689	680	396
2.21	339	654	648	700	697	408
2.49	373	674	784	787	744	518
2.49	394	721	735	810	796	483
2.51	391	689	822	825	773	559
2.74	437	796	819	903	880	544
2.97	506	845	964	996	920	663
2.98	535	900	929	1013	972	631
3.26	587	943	1091	1100	1013	744
3.49	669	1071	1143	1189	1132	758
3.58	721	1079	1273	1221	1140	854
3.76	755	1161	1259	1273	1221	836
3.99	819	1201	1421	1346	1256	943
4.00	836	1207	1438	1357	1262	952
3.99	845	1247	1375	1360	1302	906
4.04	877	1270	1401	1383	1320	929
4.27	909	1325	1473	1444	1386	967
4.50	967	1366	1629	1511	1421	1079
4.53	1045	1398	1629	1522	1438	1114
4.76	1106	1459	1684	1595	1508	1163
4.91	1143	1502	1728	1644	1551	1207
4.94	1155	1514	1734	1650	1557	1218
5.00	1195	1583	1699	1679	1626	1198
5.27	1262	1661	1771	1757	1705	1259

Table C.20, continued

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
5.50	1317	1736	1846	1823	1774	1331
5.76	1398	1789	2014	1884	1817	1464
5.94	1444	1890	2017	1936	1913	1467
5.93	1441	1878	2124	1913	1878	1551
6.02	1378	2049	2208	1855	1962	1571
6.23	1346	2142	2448	1835	1997	1734
6.03	1320	2153	2370	1803	2046	1661

Table C.21. Concrete Strains, E12.

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.01	-1	-3	0	-2
0.26	-7	-9	-8	-11
0.50	-14	-14	-15	-18
0.77	-21	-21	-23	-27
0.98	-26	-28	-29	-33
1.26	-34	-34	-41	-45
1.47	-42	-42	-49	-54
1.50	-43	-43	-51	-57
1.49	-43	-43	-51	-57
1.79	-54	-54	-64	-69
1.77	-54	-54	-64	-69
1.88	-59	-60	-71	-79
1.99	-59	-70	-77	-92
2.10	-71	-84	-88	-107
2.14	-77	-88	-94	-110
2.21	-79	-89	-96	-114
2.49	-88	-100	-107	-126
2.49	-89	-100	-109	-129
2.51	-91	-101	-110	-132
2.74	-104	-106	-126	-141
2.97	-121	-114	-148	-153
2.98	-123	-118	-152	-157
3.26	-134	-127	-166	-173
3.49	-143	-139	-180	-188
3.58	-153	-144	-190	-205
3.76	-160	-147	-197	-213
3.99	-171	-150	-214	-226
4.00	-174	-149	-217	-227
3.99	-175	-150	-219	-227
4.04	-179	-150	-227	-233
4.27	-192	-156	-243	-246
4.50	-205	-161	-263	-259
4.53	-208	-161	-271	-264
4.76	-220	-170	-289	-281
4.91	-224	-174	-298	-289
4.94	-225	-175	-299	-291
5.00	-222	-178	-306	-300
5.27	-232	-188	-323	-317

Table C.21, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
5.50	-231	-195	-332	-333
5.76	-230	-205	-350	-357
5.94	-226	-209	-353	-369
5.93	-196	-212	-325	-379
6.02	-139	-230	-253	-414
6.23	-124	-248	-218	-445
6.03	-127	-195	-215	-360

Table C.22. Deflections, U10.

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
-.00	0.02	-.000	0.000	0.000	0.000
-.00	0.02	0.001	-.000	0.000	0.000
0.27	0.28	0.005	0.006	0.000	0.000
0.49	0.50	0.011	0.009	0.001	0.001
0.75	0.75	0.017	0.015	0.001	0.001
1.01	1.00	0.023	0.021	0.001	0.001
1.26	1.26	0.031	0.030	0.002	0.002
1.48	1.48	0.039	0.039	0.002	0.003
1.76	1.77	0.055	0.058	0.004	0.005
1.85	1.86	0.070	0.072	0.006	0.006
2.00	1.99	0.087	0.082	0.007	0.007
2.00	1.99	0.107	0.105	0.009	0.010
2.25	2.24	0.118	0.123	0.011	0.011
2.33	2.35	0.154	0.162	0.016	0.016
2.48	2.48	0.181	0.178	0.018	0.018
2.51	2.51	0.206	0.197	0.020	0.020
2.51	2.50	0.207	0.199	0.020	0.021
2.76	2.76	0.224	0.216	0.022	0.022
2.98	2.97	0.262	0.295	0.026	0.027
2.94	2.93	0.278	0.315	0.030	0.030
3.01	2.98	0.306	0.344	0.034	0.034
3.49	3.48	0.368	0.412	0.044	0.043
3.57	3.53	0.412	0.454	0.048	0.047
3.98	3.97	0.458	0.505	0.053	0.052
4.00	3.96	0.486	0.527	0.056	0.054
4.27	4.26	0.509	0.554	0.059	0.057
4.50	4.48	0.547	0.590	0.063	0.061
4.30	4.44	0.578	0.599	0.064	0.060
4.51	4.47	0.600	0.606	0.064	0.062
4.78	4.77	0.636	0.644	0.068	0.066
4.98	4.95	0.665	0.674	0.071	0.069
4.97	4.93	0.675	0.681	0.072	0.070
5.01	4.96	0.691	0.695	0.073	0.071
5.51	5.47	0.755	0.764	0.081	0.079
5.50	5.45	0.766	0.781	0.082	0.080
5.78	5.75	0.808	0.826	0.087	0.083
5.99	5.96	0.839	0.858	0.090	0.087
6.29	6.27	0.895	0.916	0.097	0.092

Table C.22, continued

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
6.49	6.45	0.921	0.943	0.100	0.095
6.98	6.95	1.008	1.030	0.109	0.104
6.92	6.93	1.027	1.040	0.110	0.105
7.27	7.24	1.074	1.084	0.115	0.109
7.49	7.46	1.116	1.125	0.120	0.114
7.53	7.46	1.147	1.156	0.123	0.117
7.78	7.74	1.179	1.192	0.127	0.120
7.95	7.91	1.210	1.224	0.131	0.124
7.96	7.91	1.214	1.227	0.131	0.124
7.98	7.92	1.241	1.254	0.134	0.128
8.26	8.22	1.302	1.332	0.143	0.137
8.03	7.97	1.326	1.375	0.154	0.151
7.67	7.63	1.336	1.397	0.164	0.162

Table C.23. Bar Strains, U10.

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.00	0	-46	14	-43	-90	0
-.00	0	-46	9	-43	-90	0
0.27	6	-41	9	-32	-84	0
0.49	6	-35	14	-32	-84	6
0.75	12	-35	20	-26	-78	12
1.01	17	-26	26	-26	-72	17
1.26	29	23	6	-3	29	-35
1.48	32	-20	35	-12	-46	67
1.76	38	-3	52	0	-23	127
1.85	49	3	52	12	-17	145
2.00	61	9	58	12	3	165
2.00	110	84	81	110	69	287
2.25	130	148	67	229	232	301
2.33	313	425	405	457	339	550
2.48	457	585	573	501	382	593
2.51	527	657	651	567	443	654
2.51	535	697	631	590	541	596
2.76	573	712	706	625	492	715
2.98	666	822	822	752	596	813
2.94	680	897	886	784	703	750
3.01	700	946	941	845	758	796
3.49	790	1062	1071	949	891	906
3.57	854	1120	1132	1007	964	961
3.98	946	1189	1279	1117	975	1129
4.00	1007	1279	1297	1169	1117	1108
4.27	1053	1334	1357	1233	1172	1169
4.50	1140	1418	1441	1317	1265	1250
4.30	1143	1378	1459	1302	1187	1314
4.51	1175	1404	1493	1320	1201	1331
4.78	1242	1476	1569	1401	1285	1404
4.98	1311	1589	1595	1482	1441	1395
4.97	1323	1606	1609	1493	1459	1401
5.01	1360	1632	1632	1514	1479	1421
5.51	1488	1774	1771	1661	1624	1560
5.50	1508	1791	1780	1667	1638	1566
5.78	1606	1864	1846	1757	1722	1641
5.99	1687	1881	1933	1809	1702	1771
6.29	1826	1965	2020	1913	1800	1858

Table C.23, continued

Load			Bar Strain			
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.49	1887	2055	2043	1982	1956	1849
6.98	2061	2150	2217	2127	2032	2066
6.92	2075	2199	2199	2153	2127	2008
7.27	2156	2286	2281	2240	2211	2092
7.49	2231	2315	2376	2304	2197	2226
7.53	2266	2390	2385	2344	2321	2191
7.78	2327	2457	2451	2422	2376	2257
7.95	2370	2512	2506	2480	2414	2333
7.96	2376	2518	2506	2492	2414	2338
7.98	2388	2544	2529	2526	2425	2388
8.26	2445	2587	2636	2622	2367	2596
8.03	2411	2587	2463	2657	2445	2428
7.67	2362	2524	2341	2657	2425	2289

Table C.24. Concrete Strains, U10.

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.00	1	1	1	1
-.00	2	0	0	0
0.27	-4	-3	-6	-4
0.49	-8	-8	-11	-10
0.75	-13	-13	-18	-16
1.01	-20	-17	-28	-24
1.26	-28	-23	-38	-33
1.48	-36	-29	-50	-42
1.76	-55	-43	-79	-62
1.85	-64	-46	-91	-69
2.00	-68	-52	-100	-79
2.00	-74	-55	-114	-89
2.25	-82	-62	-126	-99
2.33	-96	-67	-146	-108
2.48	-98	-74	-152	-119
2.51	-99	-77	-160	-125
2.51	-99	-78	-161	-125
2.76	-105	-84	-173	-137
2.98	-112	-80	-192	-151
2.94	-113	-78	-193	-151
3.01	-114	-78	-200	-163
3.49	-134	-91	-223	-193
3.57	-146	-96	-236	-202
3.98	-161	-110	-258	-222
4.00	-165	-113	-265	-227
4.27	-171	-121	-277	-238
4.50	-181	-129	-292	-251
4.30	-180	-129	-290	-249
4.51	-184	-133	-296	-254
4.78	-193	-142	-312	-269
4.98	-201	-148	-324	-279
4.97	-203	-150	-326	-280
5.01	-208	-152	-333	-284
5.51	-224	-166	-361	-309
5.50	-225	-166	-365	-311
5.78	-234	-175	-381	-325
5.99	-242	-179	-393	-334
6.29	-254	-187	-414	-350

Table C.24, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
6.49	-258	-192	-422	-359
6.98	-278	-203	-454	-382
6.92	-279	-201	-455	-382
7.27	-292	-204	-473	-393
7.49	-300	-207	-485	-403
7.53	-300	-209	-486	-407
7.78	-308	-217	-498	-421
7.95	-310	-220	-503	-429
7.96	-310	-221	-503	-430
7.98	-309	-218	-502	-432
8.26	-315	-223	-509	-452
8.03	-342	-95	-540	-284
7.67	-355	-67	-553	-235

Table C.25. Deflections, E10.

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
-.00	0.00	0.000	0.002	-.000	-.000
-.01	0.00	0.001	0.001	-.000	-.000
0.25	0.28	0.007	0.007	-.000	-.000
0.48	0.50	0.010	0.011	0.000	0.000
0.74	0.79	0.016	0.018	0.001	0.000
0.97	1.00	0.020	0.022	0.001	0.000
1.24	1.29	0.027	0.029	0.001	0.001
1.45	1.50	0.033	0.034	0.002	0.001
1.62	1.69	0.040	0.042	0.002	0.002
1.77	1.84	0.044	0.045	0.003	0.002
1.86	1.94	0.048	0.051	0.003	0.002
1.92	2.01	0.053	0.056	0.003	0.003
1.95	2.03	0.058	0.062	0.003	0.003
1.77	1.83	0.075	0.090	0.006	0.005
1.96	2.04	0.103	0.110	0.009	0.009
1.96	2.04	0.113	0.123	0.010	0.010
1.95	2.03	0.113	0.123	0.010	0.010
2.14	2.22	0.132	0.137	0.011	0.011
2.25	2.33	0.162	0.167	0.014	0.014
2.30	2.38	0.176	0.185	0.017	0.018
2.27	2.36	0.184	0.218	0.019	0.020
2.42	2.52	0.225	0.240	0.021	0.022
2.43	2.54	0.236	0.249	0.023	0.024
2.66	2.77	0.263	0.279	0.026	0.027
2.81	2.92	0.302	0.312	0.036	0.035
2.93	3.03	0.320	0.328	0.038	0.037
2.94	3.08	0.343	0.352	0.041	0.040
2.93	3.08	0.344	0.353	0.041	0.040
3.19	3.32	0.361	0.372	0.043	0.042
3.20	3.22	0.406	0.388	0.047	0.044
3.41	3.56	0.441	0.413	0.051	0.047
3.46	3.61	0.459	0.433	0.053	0.049
3.68	3.82	0.476	0.450	0.055	0.051
3.90	4.07	0.511	0.509	0.060	0.056
3.89	4.08	0.525	0.527	0.062	0.058
4.23	4.42	0.558	0.562	0.066	0.062
4.39	4.59	0.584	0.592	0.070	0.065
4.43	4.64	0.607	0.614	0.072	0.068

Table C.25, continued

Load		Deflection			
South (kips)	North (kips)	South (inches)	North (inches)	West (inches)	East (inches)
4.64	4.84	0.627	0.633	0.075	0.070
4.85	5.07	0.683	0.668	0.079	0.074
5.13	5.36	0.731	0.717	0.085	0.080
5.36	5.61	0.768	0.755	0.089	0.084
5.38	5.62	0.787	0.775	0.092	0.086
5.37	5.62	0.789	0.776	0.092	0.087
5.63	5.88	0.819	0.807	0.096	0.091
5.82	6.10	0.849	0.839	0.100	0.094
5.77	6.09	0.863	0.865	0.102	0.097
5.76	6.07	0.863	0.871	0.102	0.096
6.17	6.46	0.915	0.929	0.109	0.103
6.32	6.61	0.938	0.954	0.112	0.106
6.36	6.65	0.981	1.000	0.119	0.112
6.44	6.73	0.989	1.008	0.120	0.113
6.53	6.82	0.999	1.019	0.122	0.115
6.63	6.92	1.012	1.033	0.124	0.116
6.73	7.03	1.028	1.048	0.126	0.118
6.78	7.10	1.043	1.065	0.129	0.121
6.75	7.05	1.057	1.081	0.133	0.124
6.57	6.83	1.101	1.129	0.149	0.136

Table C.26. Bar Strains, E10.

Load	Bar Strain					
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.00	-12	-3	-1027	6	-35	41
-.01	-12	-3	-1027	6	-29	41
0.25	-9	43	-1953	6	55	-12
0.48	-3	43	-1977	9	55	-12
0.74	-6	9	-1027	12	-20	52
0.97	3	49	-1965	17	67	0
1.24	9	55	-1965	20	67	6
1.45	9	61	-1953	26	72	12
1.62	12	67	-1959	29	78	23
1.77	14	29	-984	38	3	75
1.86	14	41	-941	38	3	81
1.92	26	46	-941	38	9	84
1.95	43	58	-941	43	14	90
1.77	214	229	-764	43	14	90
1.96	472	498	-454	49	20	101
1.96	506	561	-1404	67	127	122
1.95	506	567	-1415	67	133	133
2.14	538	559	-368	75	64	249
2.25	564	590	-368	171	174	602
2.30	593	619	-324	214	217	680
2.27	602	666	-1311	226	307	648
2.42	637	669	-324	246	266	767
2.43	654	683	-234	269	284	799
2.66	724	755	-171	307	350	894
2.81	767	851	-1042	330	463	897
2.93	781	880	-1019	350	495	952
2.94	819	871	29	402	463	1068
2.93	819	871	-14	405	463	1068
3.19	860	946	-952	420	573	1071
3.20	874	926	29	446	518	1163
3.41	912	975	29	478	564	1233
3.46	949	1013	29	527	614	1273
3.68	987	1094	-810	544	715	1265
3.90	1059	1123	246	605	700	1398
3.89	1088	1158	289	642	738	1415
4.23	1155	1227	376	680	790	1499
4.39	1210	1325	-579	726	935	1505
4.43	1247	1334	509	781	903	1598

Table C.26, continued

Load			Bar Strain			
	One	Two	Three	Four	Five	Six
(kips)	($\mu\epsilon$)	($\mu\epsilon$	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
4.64	1288	1412	-492	802	1016	1595
4.85	1346	1444	640	862	1007	1722
5.13	1438	1531	729	949	1108	1817
5.36	1519	1644	-182	1004	1270	1846
5.38	1563	1681	-96	1051	1314	1870
5.37	1569	1687	-107	1051	1320	1878
5.63	1624	1748	3	1088	1375	1939
5.82	1690	1809	98	1140	1430	2011
5.77	1710	1835	145	1187	1461	2011
5.76	1707	1794	1213	1192	1383	2063
6.17	1803	1887	1346	1273	1482	2168
6.32	1852	1933	1433	1314	1531	2217
6.36	1913	1977	1519	1418	1635	2260
6.44	1927	1988	1566	1435	1647	2278
6.53	1945	2008	1566	1450	1661	2301
6.63	1968	2026	1609	1467	1684	2327
6.73	1997	2089	538	1493	1803	2304
6.78	2017	2075	1696	1528	1745	2370
6.75	2003	2121	599	1522	1852	2330
6.57	1959	2153	567	1514	1968	2275

Table C.27. Concrete Strains, E10.

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
-.00	3	0	3	1
-.01	3	1	3	1
0.25	-3	-4	-4	-4
0.48	-8	-6	-9	-8
0.74	-13	-13	-17	-16
0.97	-18	-18	-22	-22
1.24	-23	-25	-30	-29
1.45	-28	-30	-36	-36
1.62	-33	-34	-43	-43
1.77	-37	-38	-48	-47
1.86	-39	-40	-52	-50
1.92	-41	-42	-54	-51
1.95	-42	-43	-55	-54
1.77	-40	-42	-53	-51
1.96	-44	-46	-60	-58
1.96	-44	-48	-60	-61
1.95	-43	-49	-61	-61
2.14	-49	-53	-65	-65
2.25	-54	-58	-72	-72
2.30	-68	-59	-91	-94
2.27	-78	-57	-103	-107
2.42	-84	-57	-110	-120
2.43	-85	-50	-112	-125
2.66	-92	-53	-120	-139
2.81	-110	-54	-157	-151
2.93	-114	-58	-162	-156
2.94	-119	-57	-170	-161
2.93	-120	-57	-170	-161
3.19	-125	-59	-178	-171
3.20	-126	-57	-178	-169
3.41	-133	-61	-189	-178
3.46	-136	-59	-193	-179
3.68	-141	-63	-201	-187
3.90	-149	-64	-212	-194
3.89	-151	-64	-216	-196
4.23	-159	-71	-228	-210
4.39	-164	-76	-236	-218
4.43	-167	-79	-239	-222

Table C.27, continued

Load	Concrete Strain			
	West	East	West	East
(kips)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)	($\mu\epsilon$)
4.64	-171	-85	-248	-232
4.85	-177	-89	-257	-241
5.13	-188	-100	-273	-255
5.36	-196	-108	-284	-268
5.38	-201	-111	-290	-268
5.37	-202	-114	-290	-269
5.63	-208	-120	-302	-282
5.82	-217	-126	-313	-292
5.77	-222	-128	-315	-294
5.76	-222	-128	-316	-294
6.17	-238	-141	-340	-312
6.32	-244	-141	-347	-319
6.36	-257	-151	-365	-319
6.44	-259	-153	-370	-321
6.53	-261	-155	-373	-325
6.63	-263	-156	-378	-328
6.73	-263	-159	-383	-330
6.78	-263	-157	-387	-330
6.75	-257	-156	-380	-332
6.57	-167	-66	-321	-226

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