ANCHORAGE AND DEVELOPMENT OF TWO-BAR BUNDLES IN ONE AND TWO LAYERS

by

WEIXIONG CHEN, B.S.

Presented to the Faculty of the Graduate School of
The University of Texas at Austin
in Partial Fulfillment
of the Requirements
for the Degree of

MASTER OF SCIENCE IN ENGINEERING

THE UNIVERSITY OF TEXAS AT AUSTIN

August 1994

ABSTRACT

ANCHORAGE AND DEVELOPMENT OF TWO-BAR BUNDLES IN ONE AND TWO LAYERS

by

WEIXIONG CHEN

The University of Texas at Austin

SUPERVISOR: James O. Jirsa

Bond strength tests of two-bar bundles were conducted using beam specimen in which partially unbonded bars were used. Variables included number of layers, amount of transverse reinforcement, casting position, and shear along the development zone. An emphasis was placed on the applicability of previous established bond strength equation to the case of two-bar bundles. The bond strength equation for two-bar bundles was proposed and the bond failure mechanism for bundled bars in one and two layers was explained.

ANCHORAGE AND DEVELOPMENT OF TWO-BAR BUNDLES IN ONE AND TWO LAYERS

APPROVED:

James O. Jirsa

Dr. James O. Jirsa

John E. Breen

Dr. John E. Breen

DEDICATION

To my Wife, Parents, Brother, and Sister for their love and support my entire life.

ACKNOWLEDGEMENT

Research was conducted at the Phil M. Ferguson Structural Engineering Laboratory at The University of Texas at Austin. Funding was provided by Texas Department of Transportation.

The author would like to express his great thanks to Dr. James O. Jirsa for guidance, friendship, and encouragement throughout author's study in UT. This thesis could not be finished without his help. In addition, special thanks to Dr. John E. Breen for being my second reader.

Sincere thanks to the staff at the Ferguson Structural Engineering Laboratory for their help throughout this project. Without the help of Wayne Fontenot, Blake Stasney, Wayne Little, Pat Ball, Ray Madonna, and Ryan Green, the author is probably still chipping concrete right now. Thanks are also given to April Jenkins, Laura Golding, and Sharon Cunningham for their kindness.

The author is owning to the friendship and help from the friends working here. Especially, my deepest thanks should firstly be given to Daniel B. Grant and Jeffrey S. Schmitz for their help and assistance throughout the project.

Finally I need to say that none of this would have been possible without the care and support from my wife and my family.

Weixiong Chen

August, 1994

TABLE OF CONTENTS

FIGURE LIST

TABLE LIST

CHAPTER 1
INTRODUCTION

1.1 Problem Statement

In the design of reinforced concrete structures, particularly those for supports of bridges (pier caps, bents, etc.), a great deal of reinforcement must be placed in areas where available space is limited. Congestion of reinforcement and difficulty in placing and consolidating concrete often result. One way to solve this problem is to place the reinforcing bars in bundles. Another solution is to arrange the bars in multiple layers. The clear spacing between reinforcement groups will be increased considerably by using bundled bars. Larger spacing will greatly facilitate concrete placement and insertion of spud vibrators and hence improve the quality of the concrete.

Due to the complicated mechanism of force transfer between reinforcement and concrete, and the non-uniformity of concrete, current specifications for bond strength in building codes are based on experimental data. A great many experiments have been done to study anchorage, development and splicing of deformed bars. However, most of the research involved was done by testing non-bundled bars in the tension zone of structural members. While such tests provide much useful data about anchorage and development of deformed reinforcement, the results may not represent situations where bundled bars or multiple layers of reinforcement are used. Codes are silent about the anchorage of a group of bundled bars and of bars in multiple layers, or provide vague guidance without experimental support.

The purpose of this study is to examine experimentally the anchorage strength of two-bar bundles in one or two layers, and to evaluate the applicability of equations for non-bundled bars to two-bar bundles.

1.2 Project Background

The test program is the first portion of a project on anchorage and development of groups of reinforcing bars, which is sponsored by Texas Department of Transportation (TxDOT). There are many cases where bundled bars are used in one layer or multiple layers in TxDOT projects. The most typical applications are the reinforcement for inverse T-beams or bents in highway bridges, as shown in Figure 1.1 and Figure 1.2. The research work is based on typical TxDOT designs in which two-bar bundles are placed in one layer or two layers. In order to determine bond strength, the specimens were designed to fail in bond before the reinforcement yielded.

1.3 Object and Scope

The primary object of this study was to examine the effects of placing reinforcement in two-bar bundles, and in one or two layers, on bond strength and development length. Emphasis was placed on evaluating the applicability of previously established equations for estimating bond strength, which are based on tests of non-bundled bars in single layer, to cases where bars are placed in bundles and multiple layers. In addition, the bond failure mechanism of bundled bars placed in one layer and two layers was evaluated.

Pull-out tests which were conducted using beam specimens included a study of the effects of following variables:

(1) Arrangement of bars: one layer and two layers
(2) Casting position: top cast (more than 12 in. of fresh concrete below the bars) and bottom cast
(3) Effect of transverse reinforcement
(4) Effect of shear acting along the anchorage zone

The other variables such as concrete strength, diameter of reinforcement, anchorage length, face cover and clear spacing were kept constant.

The behavior of the specimens is described in terms of failure mode, crack pattern and bar stresses at various levels up to bond failure. The results provide data for calculating bond strength of two-bar bundles arranged in one layer and two layers. Also, the results provide guidance for designing the next stage of the research program.

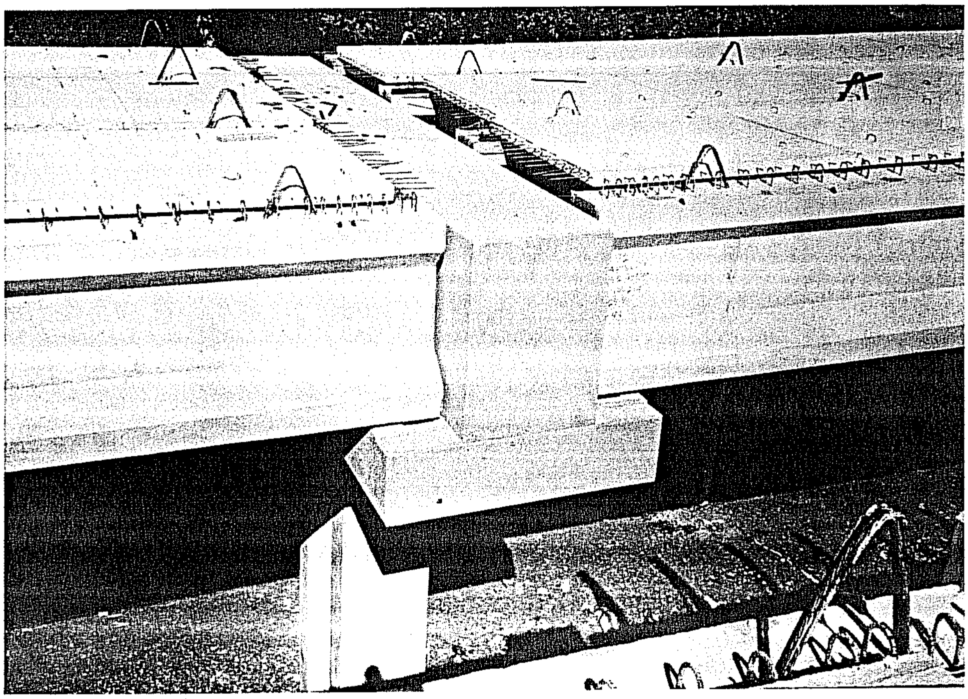


Figure 1.1 Inverse T-Beam

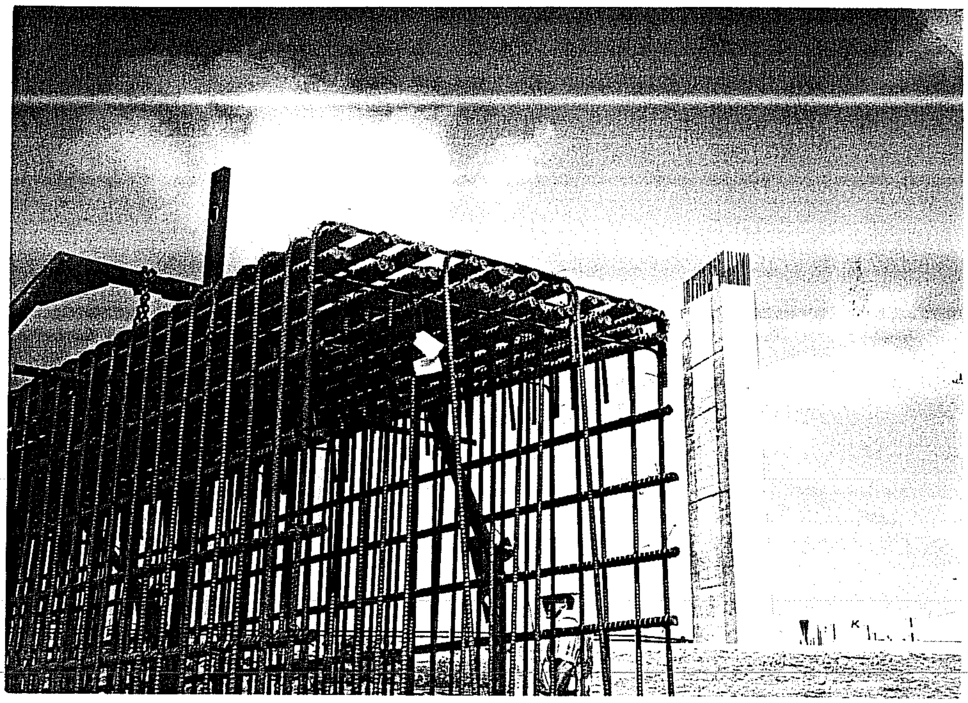


Figure1.2 Cage of Bent Cap

Figure 1.1 Inverse T-Beam

Figure1.2 Cage of Bent Cap

CHAPTER 2
BOND FAILURE HYPOTHESIS

2.1 Stress Transfer Mechanism

Bond stresses are assumed to represent the average shear stress between embedded reinforcement and the surrounding concrete. Early practice in reinforced concrete design involved plain bars for reinforcement. For plain bars, the bond strength is controlled mainly by a combination of chemical bond between the cement paste in concrete and the bar surface, and friction between the reinforcement and adjacent concrete. Together chemical bond and friction provide very little bond strength. For high strength reinforcement or large diameter bars, adhesion and friction usually can not provide enough anchorage force to yield the bar within a reasonable anchorage length. For this reason, deformed reinforcement is used. In addition to adhesion bond and friction, there is mechanical interlock between concrete and the lugs on the deformed reinforcement. Most of the bond strength is provided by mechanical interlock. Although the deformed bar has higher bond strength, there is a greater tendency for failure to be produced by concrete spitting between the bars or in the cover.

Mechanical interlock is determined by many parameters, including the height, the inclined angle and the spacing of the lugs on bars; the concrete strength, and the amount of concrete or transverse steel surrounding the bars. Since concrete is a brittle and non-uniform material, stress transfer between reinforcement and concrete is not uniform. As a result, the average bond stress rather than the bond stress at a particular point along the embedded bar is used to assess the performance. By assuming bond stress is uniform along the anchorage length, the average bond stress can be calculated by equating the tensile force in the bar to the bond force acting on the cylindrical surface area of the anchored bar. The surface area is based on the nominal bar diameter, ignoring the extra surface area and bearing resistance provided by the lugs, as indicated in Equation 2.1:

$$T = \frac{\pi d_b^2}{4} f_s = l_s \, (\pi d_b) \, u \tag{2.1}$$

where: T = the tension force on bar

d_b = diameter of bar

f_s = stress on bar

u = average bond stress along the anchorage length

Rearranging equation 2.1

$$u = \frac{f_s\, d_b}{4\, l_s} \tag{2.2}$$

If the tensile force on the bar is called active force, the reactive force should be provided by the concrete on the lugs of the bar. This reacting force, N, is inclined at an angle β to the axis of the bar as shown in Figure 2.1. The angle of inclination "β" has been found to vary from 45° to 80°, depending on the rib geometry. While "U", the horizontal component of the inclined reactive force "N", balances the tensile active force on bar, the vertical component "U' ", like water pressure, produces a radial pressure on the concrete cylinder. The radial pressure is balanced by the tensile stress in the concrete surrounding the bar. As shown in Figure 2.2., the radial pressure can be considered as an internal pressure acting against a thick-walled cylinder having an inner diameter equal to the bar diameter d_b and thickness parameter C. As shown in Figure 2.3, C is the smallest of 1) the thickness of face cover C_b ; 2) half the clear spacing between the adjacent bars C_s; 3) the side cover S'/2 [1]. Depending on the concrete strength and the parameter C, the concrete failure can be classified as

(1) pull-out failure: reinforcement lugs shear off surrounding concrete, if C is large

(2) splitting failure: concrete cover fails in tension and spalls

The mode of splitting failure depends on the values of C_b and C_s as shown in Figure 2.3. With $C_b > C_s$, horizontal splitting develops at the plane of the bars, and this is termed "side-split-failure". With $C_s > C_b$, longitudinal cracks through the cover form before splitting through the plane of the bars. Such a failure is termed a "face-and-side split failure." With $C_s >> C_b$, a "V-notch failure" occurs with longitudinal cracking followed by inclined cracking. [1]

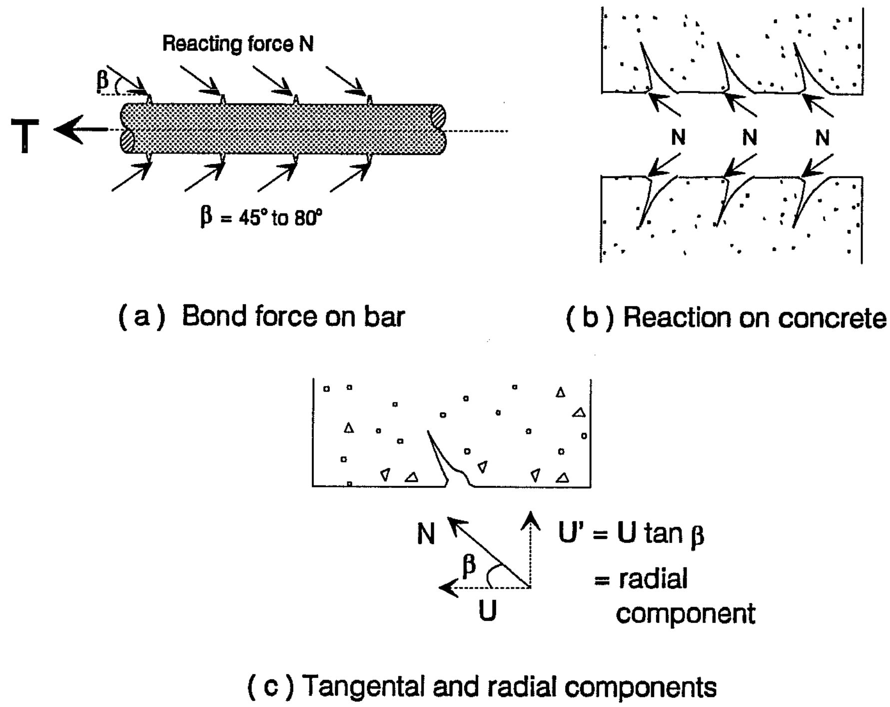
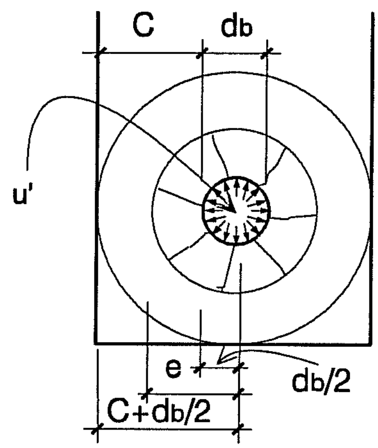


Figure 2.1 **Forces between deformed bar and concrete (Ref. 5)**



C = The smallest of :

(1) The thickness of face cover C b
(2) Half the clear space between the next adjacent bar. C s
(3) The side cover C d

Figure 2.2 **Radial pressure acting on a thick-walled cylinder with inner diameter equal to d b and a thickness equal to C (Ref. 5)**

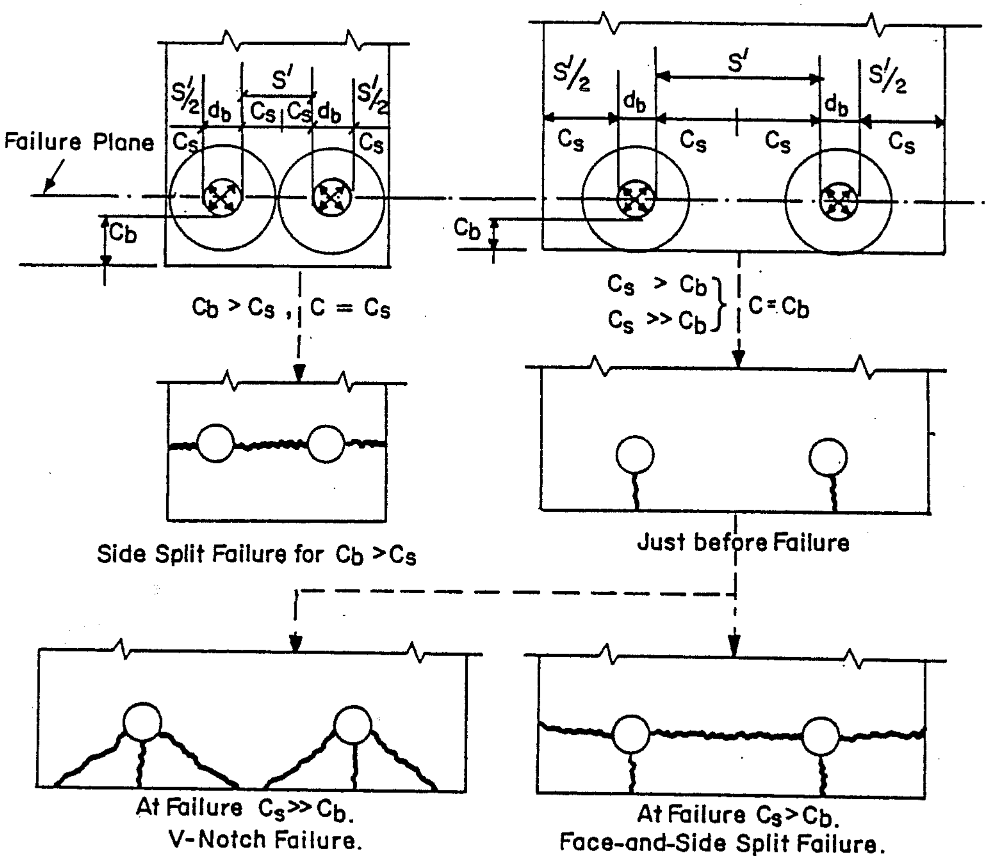


Figure 2.3 Failure patterns of deformed bars (ref. 1)

When bars are bundled, the "effective" surface area transferring bond may be changed. No data was found in literature for bond stresses of bundled bars or for the mechanism of bond failure in two layers of reinforcement.

2.2 Previous Research

2.2.1 Non-bundled Bars in One layer

There is much test data in the literature dealing with the bond strength of non-bundled bars in one layer. The following factors are considered to influence the bond strength: concrete tensile strength, diameter of bar, embedded length, thickness of side or face concrete cover, transverse reinforcement confinement, and casting position. Orangun, et al, derived empirically, using nonlinear regression methods, an equation to estimate the bond strength based on a number of well-documented tests. In the Orangun formula, the bond stress is expressed as a function of bar imbeded length, concrete tensile strength, bar diameter, thickness of side or face concrete cover, transverse reinforcement and casting position. [1]

The total bond strength may be regarded as the combination of that due to concrete and transverse reinforcement.

$$u_{cal} = u_c + u_{tr} \tag{2.3}$$

where u_c represents the bond stress contributed by the concrete, and u_{tr} represents the bond stress contributed by the confinement of transverse reinforcement.

By using non-dimensional parameters $u/\sqrt{f_c'}$, c/d_b, and d_b/l_s, the average bond strength contributed by concrete can be expressed by formula (2-4):

$$\frac{u_c}{\sqrt{f_c'}} = 1.2 + 3\,\frac{c}{d_b} + 50\,\frac{d_b}{l_s} \tag{2.4}$$

Transverse reinforcement increases the bond strength by the following factor:

$$K_{tr} = \frac{u_{tr}}{\sqrt{f_c'}} = \frac{A_{tr}\, f_{yt}}{500\ s\ d_b} \tag{2.5}$$

The total bond stress can be expressed in following:

$$u_{cal} = \left(1.2 + 3\ \frac{c}{d_b} + 50\ \frac{d_b}{l_s} + \frac{A_{tr}\, f_{yt}}{500\ s\ d_b} \right) \sqrt{f_c'} \tag{2.6}$$

The above formulae are subjected to the following limitations:

a) $c/d_b \leq 2.5$

b) $A_{tr}f_{yt}/500sd_b \leq 3.0$

c) for bars in the top cast position, Equation (2.6) should be divided by a factor of 1.3

Where

u_{cal} = calculated ultimate bond stress, psi

u_c = portion of bond stress contributed by the concrete cover, psi

u_{tr} = portion of bond stress contributed by the transverse reinforcement, psi

K_{tr} = index of the strength provided along the anchored bars by the transverse reinforcement

C = minimum thickness of face cover and half spacing of adjacent bars, in.

d_b = bar diameter, in.

A_{tr} = area of transverse reinforcement crossing the splitting plane through the anchored bars, in^2

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

s = spacing of transverse reinforcement, in

f_c' = concrete compressive strength, psi

For transverse reinforcement to be effective in improving the anchorage strength, the legs of transverse reinforcement should be adjacent to the longitudinal reinforcement and normal to the potential splitting cover. The "A_{tr}" term can be regarded as the average effective transverse area for a single anchored bar.

$$A_{tr} = \frac{\Sigma a_b}{n_s} \tag{2.7}$$

where n_s = number of the enclosed bars in the section.

a_b = area of transverse reinforcement per leg

Equation (2.6) gives a good estimation of the relationship between anchorage strength and other parameters. Setting equation (2.2) equal to equation (2.6), the relation between the anchorage length l_s and the bar stress f_s can be found:

$$l_s = \frac{d_b \dfrac{f_s}{4\sqrt{f_c'}} - 50}{1.2 + 3\dfrac{c}{d_b} + \dfrac{A_{tr}\, f_{yt}}{500\; s\; d_b}} \tag{2.8}$$

Rearranging Equation 2.8, f_s can be expressed in terms of l_s as following:

$$f_s = \frac{4\; l_s}{d_b} \left(1.2 + 3\frac{C}{d_b} + 50\frac{d_b}{l_s} + \frac{A_{tr}\, f_{yt}}{500\; s\; d_b} \right) \sqrt{f_c'} \tag{2.9}$$

2.2.2 Bars in Multiple Layers

There is little information in the literature covering the bond behavior of multiple layers of bars. In Reference 6, some tests are reported on the anchorage behavior of multiple layers of reinforcement at beam end support. As shown in Figure 2.4, the dimension of the specimen and the arrangement of the reinforcement can be interpreted to more closely represent a vertical layer of reinforcement rather than two horizontal layers of reinforcement. The horizontal spacing between the bars was 250 mm (10 in), the vertical spacing was only 30 mm (1 $^3/_{16}$ in). The distance between two groups of bars was so large that the interaction between the groups was small. It is likely that the two bars on each side worked as two single bars in a vertical orientation. The test results showed that most specimens failed due to the splitting of corner concrete.

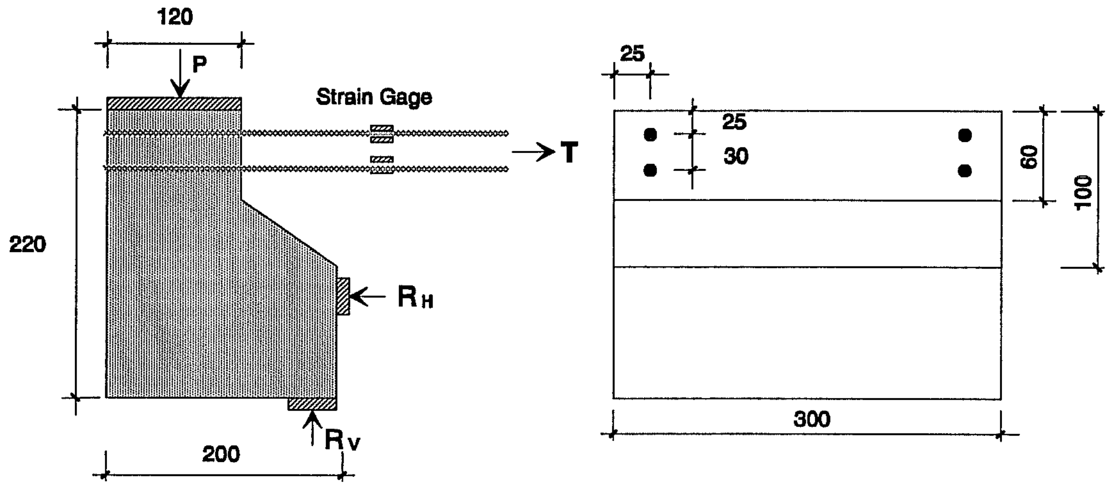


Figure 2-4: **Specimen of Beam End Support (Ref. 6)**

2.2.3 Bundled bars

There is little test data on the behavior of bundled bars. One study is reported in Reference 7 in which pairs of specimens of large beams with conventionally spaced and bundled longitudinal bars were tested. The bundles consisted of groups of four No. 6, four No. 8, and three No.9 bars, some in contact and some with space between bars as shown in Figure 2.5. Pairs of beams were compared with respect to width of flexural cracks, steel stress distribution, deflection, and ultimate strength. No significant difference in behavior or ultimate strength was found for bundled bars as compared to spaced bars.[7]

2.3 Current AASHTO and ACI318-89 Code Provision Regarding the Bundled Bars

The required development length for bundled bars is the same in the AASHTO-89 and ACI 318-89 codes. Both specify that the development length for individual bars within a bundle should be increased 20 percent for the three-bar bundle, and 33 percent for four-bar bundle. The unit of bundled bars shall be treated as a single bar with a diameter derived from the equivalent total area when choosing the appropriate modified factors. There is no special requirement for two-bar bundles because the surface bonding area between the bundled bars and surrounding concrete does not change in the case of two-bar bundle. The modification factor of development length for top cast bars is 1.4 in AASHTO and 1.3 in ACI code.

Neither code provides comments or provisions regarding the development length for multiple layers of bundled bars.

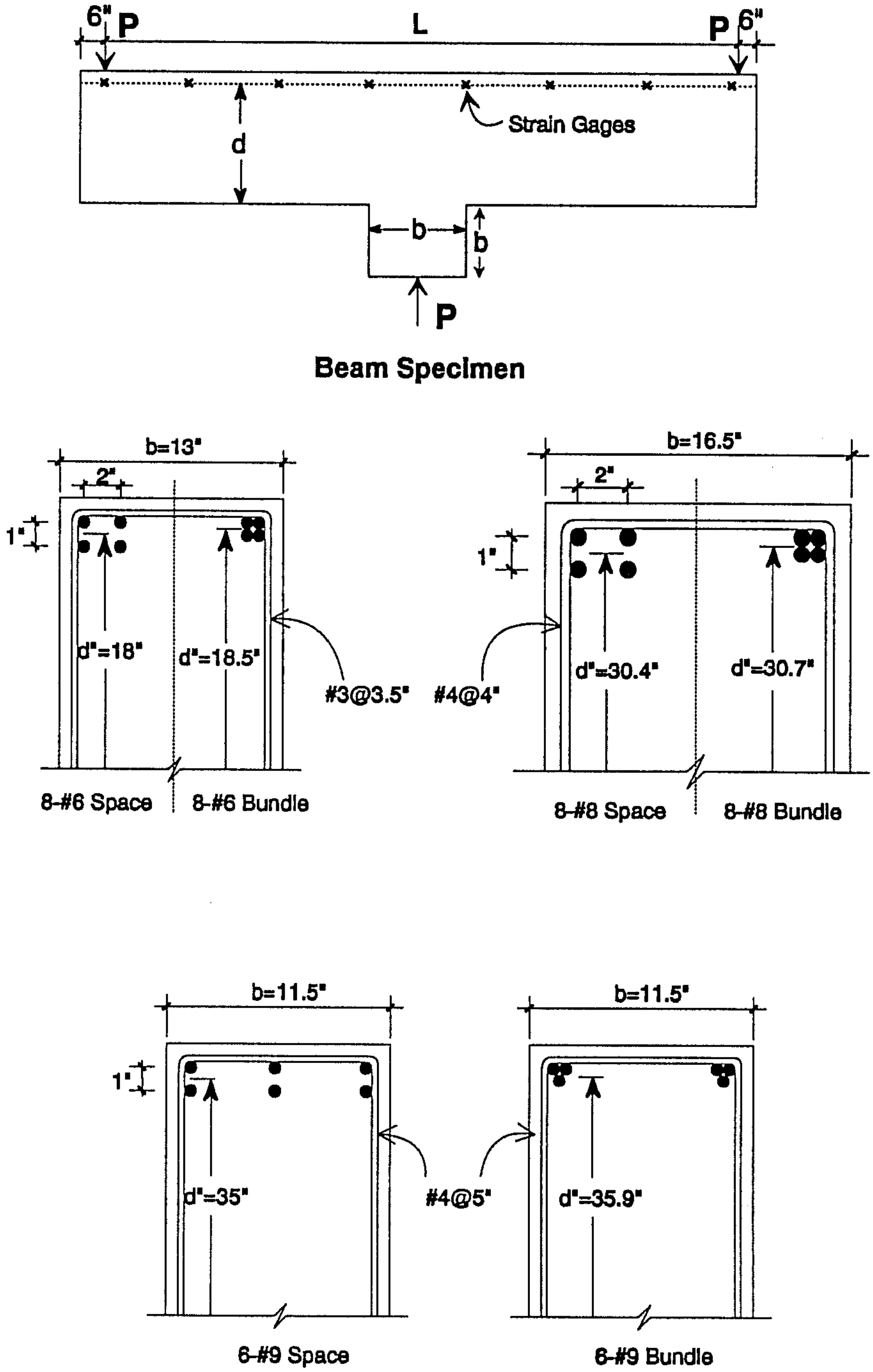


Figure 2.5 **Beam specimen for comparable bundled bar test (Ref. 7)**

CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Introduction

Tests were conducted using beam specimen in which partially unbonded bars were used to permit determination of the bond strength of two-bar bundles in one layer and two layers. The variables included the number of layers, amount of transverse reinforcement, casting position, and shear along the development zone. A total of three beams were built, and each was initially designed to permit four separate tests. In the first beam, there was no shear applied to the test region and a strip of plywood (Figure 3.8) was used to isolate the test region from the rest of the beam. It was found that the plywood changed the failure mechanism in the test region and the test results did not reflect normal bond behavior as will be discussed in section 3.3. A second beam was identical to the first (Figure 3.17), but with a teflon sheet instead of plywood. Some additional tests were conducted on the second beam to acquire data that was to have been obtained from the first beam. In the third beam, shear was imposed on test region.

The variables are discussed in 3.2 and summarized in Table 3.1. The details of the beam specimens are presented in 3.3. Material properties are included in 3.4, and the test setup is described in 3.6.

3.2 Variables

The main purpose of this program was to study the bond behavior of two-bar bundles placed in one and two layers. Previous research on the bond of one layer of non-bundled bars showed that the bond strength was affected by several parameters. The most important parameters were concrete strength, diameter of the rebar, confinement by transverse reinforcement, concrete cover or clear spacing between bundles, and casting position. In this program, the concrete strength ($f_c' \approx 3500$ psi), diameter of reinforcement ($d_b = {}^3/_4$", #6 bar), concrete face cover ($C_c = 1$") and clear spacing of rebars ($2C_s = 2\,{}^5/_8$ ") were kept constant, and the effect of the number of layers, the confinement of transverse reinforcement, casting positions, and influence of shear were examined.

Table 3.1 Details of Test Specimen

Test No.	Specimen No.	Anchorage Length L_d (in.)	Clear Spacing C_s (in.)	Face Cover C_b (in.)	Layer Spacing C_d (in.)	Concrete Age (Day)	Concrete Strength (ksi)	Yield Strength (ksi)		Transverse reinforcement			Casting position
								Longit. Bar	Trans. Bar	No.	Legs	space (in)	
8	1-24-NS-T	24	$2^5/_8$	1		42	2.9	61.2	66.1				Top
12	1-24-NS-B	$23^1/_2$	$2^5/_8$	$1^1/_8$		49	4.2	61.2	66.1				Bottom
9	1-16-NS-T	16	$2^5/_8$	1		78	2.9	61.2	66.1	#4	4	8	Top
13	1-16-NS-B	$15^3/_8$	$2^5/_8$	$1^1/_8$		39	2.5	61.2	66.1	#4	4	8	Bottom
11	2-24-NS-T	24	$2^5/_8$	$1^1/_2$	$1^1/_4$	43	4.2	61.2	66.1				Top
6	2-24-NS-B	24	$2^5/_8$	1	$1^1/_4$	47	2.9	61.2	66.1				Bottom
14	2-16-NS-T	15	$2^5/_8$	1	$1^1/_4$	46	2.6	61.2	66.1	#4	4	8	Top
10	2-16-NS-B	16	$2^5/_8$	1	$1^1/_4$	83	2.9	61.2	66.1	#4	4	8	Bottom
7	1-24-S-T	24	$2^5/_8$	1		37	2.6	61.2	66.1				Top
8	1-16-S-T	16	$2^5/_8$	1		41	2.6	61.2	66.1	#4	4	8	Top
15	2-24-S-B	24	$2^5/_8$	1	$1^1/_4$	64	2.7	61.2	66.1				Bottom
16	2-16-S-B	16	$2^5/_8$	1	$1^1/_4$	66	2.7	61.2	66.1	#4	4	8	Bottom
1	1-24-NS-B*	24	$2^5/_8$	1			3.7	61.2	66.1				Bottom
3	1-16-NS-B*	16	$2^5/_8$	1			3.7	61.2	66.1	#4	4	8	Bottom
2	2-24-NS-T*	24	$2^5/_8$	1	$1^1/_4$		3.7	61.2	66.1				Top
4	2-16-NS-T*	16	$2^5/_8$	1	$1^1/_4$		3.7	61.2	66.1	#4	4	8	Top

Note: * Tests without teflon sheet backing are listed just for information.

For easy reference, four labels are used to denote tests as explained in Figure 3.1.

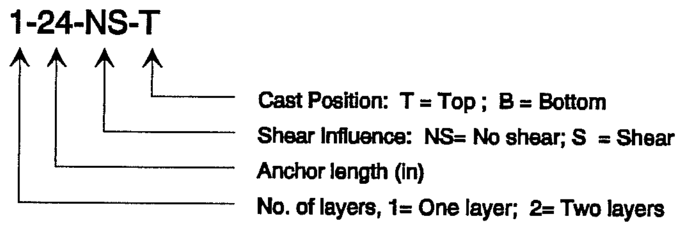


Figure 3.1 Notation of test numbering system

3.2.1 Layers

In the test of a single layer of bars, there were ten bars in the section divided into five two-bar bundles. In the test with two layers, there were twenty bars with ten bars in the outer layer and another ten bars in the inner layer. The ten bars in each layer were divided into five two-bar bundles. The center to center distance between the two layers was 2 inches. With other variables hold the same, the bond strength of bundled bars in one and two layers could be compared.

3.2.2 Casting Position

In the AASHTO code it is specified that if more than 12 inches of fresh concrete is cast below the reinforcement, this reinforcement is defined as "top cast reinforcement". It is well documented that the bond strength is reduced for bars in the top cast position. To investigate the effect of casting position on bundles, it was necessary to perform identical tests for each case being examined; one cast at the top of a beam, and a duplicate cast at the bottom.

3.2.3 Confinement of Transverse Reinforcement

Comparable tests, one with transverse and other without transverse reinforcement, were constructed to provide a means of evaluating the effects of transverse reinforcement.

3.2.4 Shear

Four tests had shear imposed on the test region. The geometry and reinforcement in these tests were identical to those specimens without shear. The intent was to compare anchorage strength for companion tests with and without shear.

3.3 Specimen Design

There are many variables affecting the bond behavior of bundled bars. To reflect the actual behavior of bundled bars in the field and to minimize the number of test specimens, the beam dimensions and the arrangement of reinforcement were based on a TxDOT bent cap design. To meet the geometry and force limitation imposed by the test floor in Ferguson Structural Laboratory, a half scale specimen was used.

For a typical application in which bundled bars were used, it was necessary to load many bars simultaneously. To avoid having to develop a complicated loading system, each specimen was designed as a beam with some lengths of reinforcement unbonded in certain regions. The bundled bars were placed in light gage tubes which separated the bars from the surrounding concrete. The purpose of the bundles being unbonded was to produce a condition such that force in the bars was transferred only in the anchorage length.

For the tests without shear acting on the anchorage region, the loading is shown in Figure 3.2. If the beam is loaded as shown, there is no shear on the test region and the maximum moment will be located at the support. After the beam cracks above the support, all the bundled bars were subjected to the same deformation. The tensile force in the bars over the support is transferred to the bundled bars in the active test region; in the other direction, the middle portion of the beam (3-feet long), the bars were bonded and anchored to develop the tensile force. Using this arrangement all the bundled bars in the active test region are loaded simultaneously.

For the tests with shear acting on the anchorage region, the test set-up is the same as that without shear except the test region is moved to a location between the loading point and support (Figure 3.3). The test region was under a constant shear when the beam was loaded.

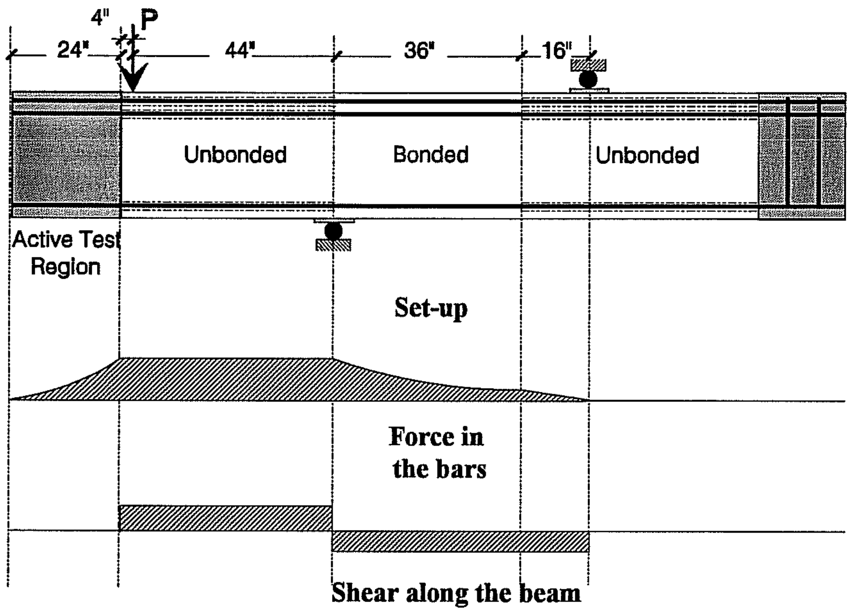


Figure 3.2 **Test setup without shear**

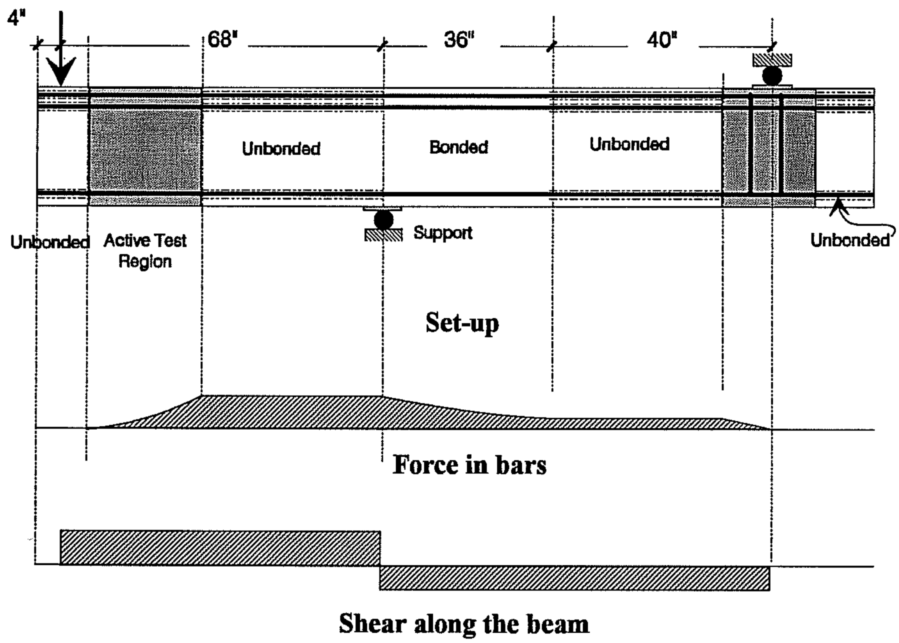


Figure 3.3 **Test setup for the test with shear**

All three beams were 22 inches wide and 30 inches deep. The beam cross section and the dimensions of cover and clear spacing are shown in Figure 3.4. More than 25 inches of concrete were cast below the upper bars in the cage, so the upper bars are top cast bars according to AASHTO requirements (more than 12 inches of concrete cast below). The face cover was 1 in. thick, the side cover was 2 in., and the clear spacing of longitudinal bars was $2^5/_8$ in. The total length of the beam was 15 feet.

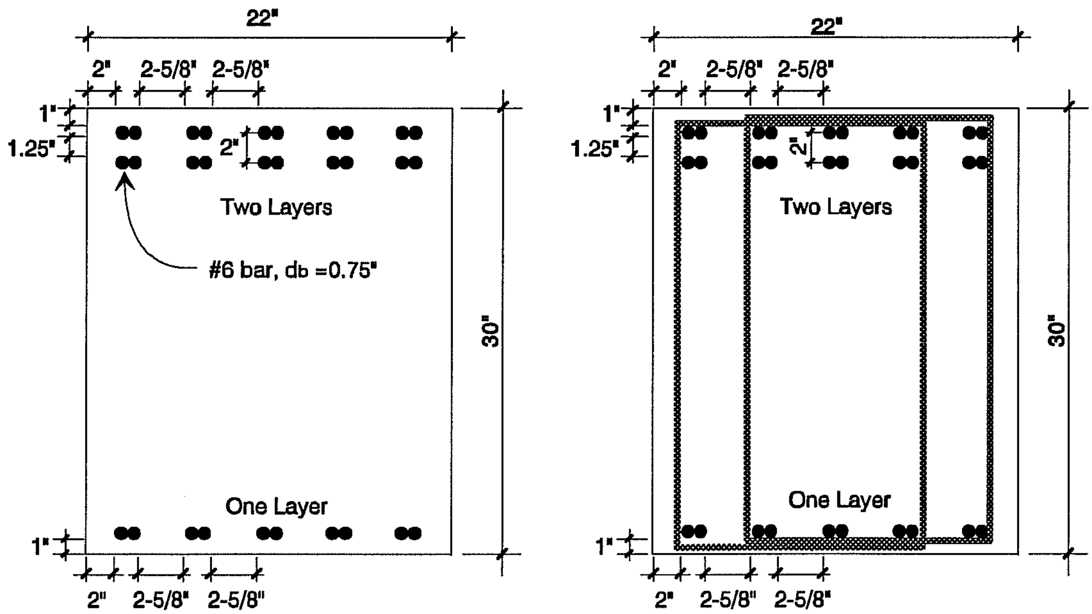


Figure 3.4 **Cross Section of the Beam**

The stress in the reinforcement was measured by strain gages. The strain gages on the longitudinal reinforcement were attached about 3 inches from the boundary of the active test region and unbonded region as shown in Figures 3.5, 3.6, and 3.7. The strain gages were inside the steel tubes and were isolated from the concrete. In this way the strain gages were well protected and the gaged bars were reused in some cases.

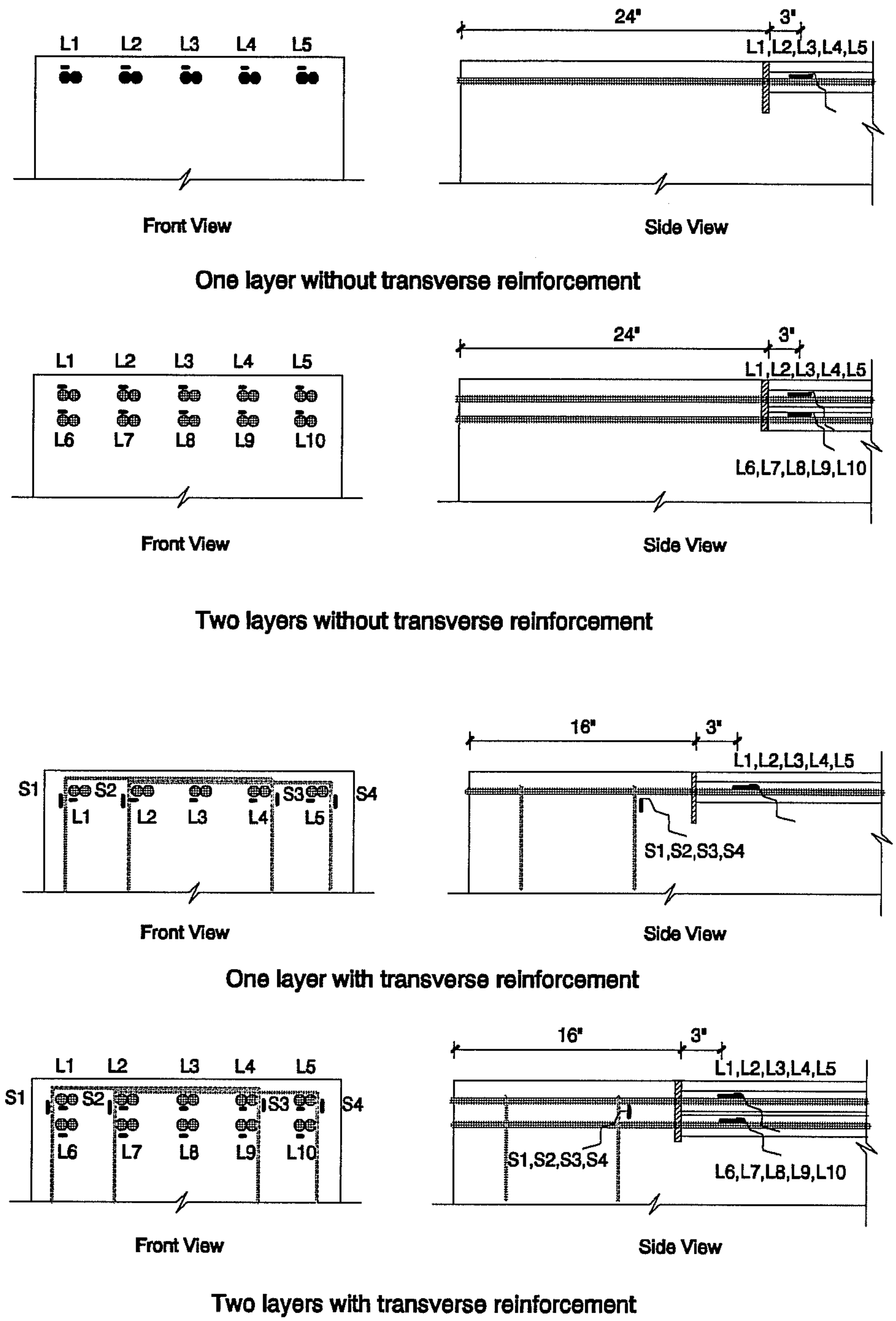


Figure 3.5 Position of strain gages in specimen #1

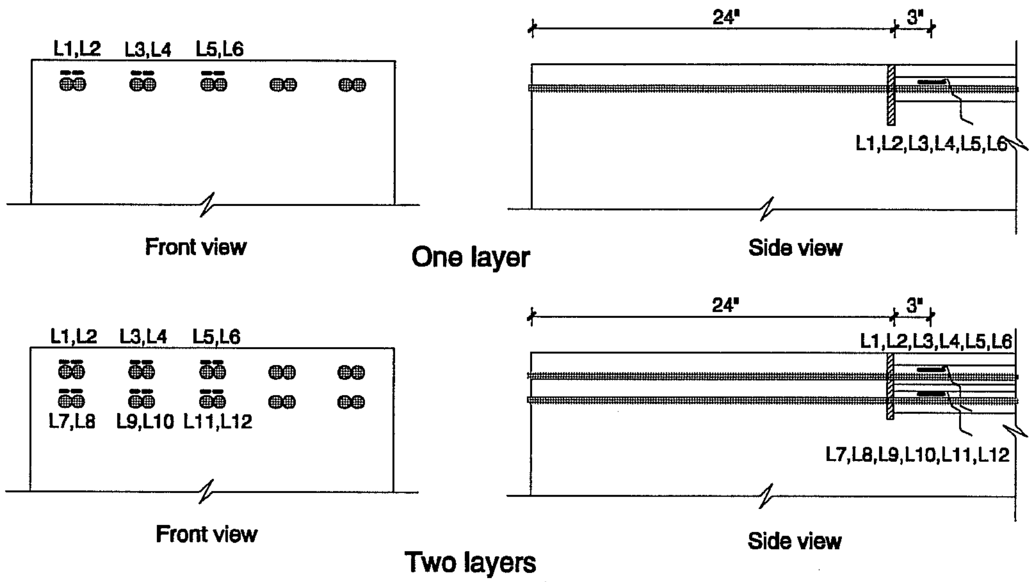


Figure 3.6 **Position of strain gages for test without transverse reinforcement (specimen #2, #3)**

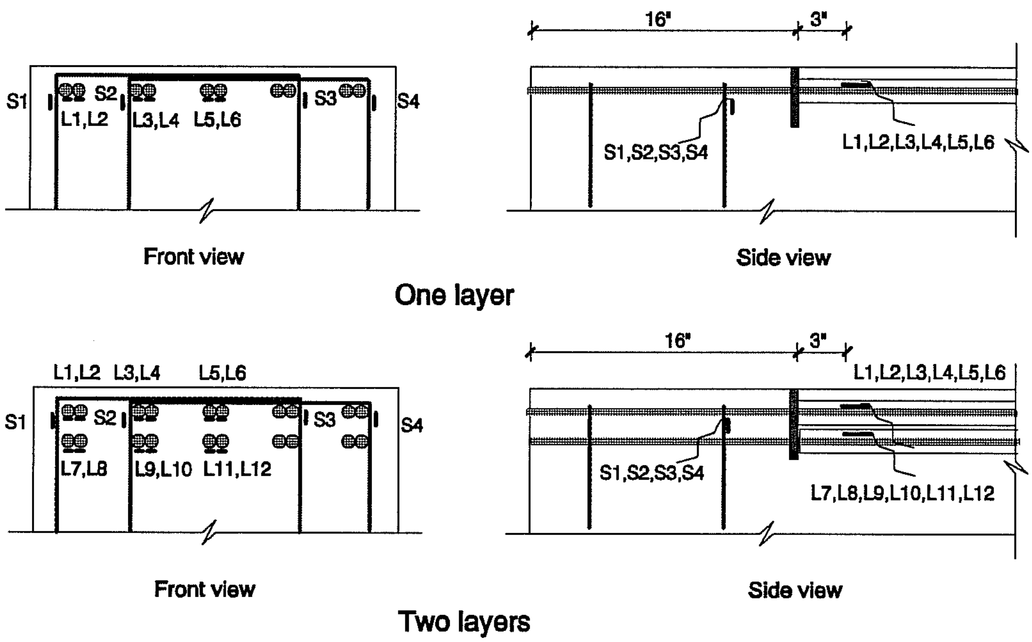


Figure 3.7 **Position of strain gages for test with transverse reinforcement (Specimen #2,#3)**

3.3.1 Specimen #1 (without Shear)

Initially, this specimen was designed to provided four tests without shear. However the plywood strip changed bond behavior in the test region. As a result, these four tests were repeated using Specimen #2 as discussed later.

The details of Specimen # 1 are shown in Figure 3.8, two layers of bundled bars were at the top of the beam and a single layer of bundled bars was on the bottom. 1" × 2 " light gage steel tubes enclosed bundled bars in two regions. The stress in the rebars in this region was constant. To ensure that a bond failure occurred before the bars yielded, the anchorage length was less than specified by AASHTO. For the case without transverse reinforcement, the bonded length was reduced from 25 inches to 24 inches; for the case with ties, the length was reduced from 17.8 inches to 16 inches. As shown in Figure 3.8, the left end of the beam is the test region without stirrups; and the right end is the test region with stirrups.

To reduce the confining effect of loading on splitting in the test region and to clearly define the anchorage length, the test region had to be separated from the unbonded region. A piece of plywood was placed at the boundary between the test region and the unbonded region.

3.3.2 Testing of Specimen #1

Four tests were carried out on this specimen: one layer of bottom cast bars with and without transverse reinforcement, two layers of top cast bars with and without transverse reinforcement. All four tests had no shear acting on the test region. For easy reference, these four tests are denoted by the following symbols which are explained in Figure 3.1.

1-24-NS-T*	1-16-NS-T*
2-24-NS-B*	2-16-NS-B*

Because plywood was much softer than concrete, it formed a soft layer between the test region and bearing concrete. Before testing, the outside part of the plywood was chipped off. This left a notch in the concrete cover (Figure 3.9) and permitted the concrete cover to rotate when the cover split, as shown in Figure 3.10. These factors changed the test conditions and lowered the bond

Table 3.2 Comparison of Test Results with and Without Teflon Sheet

Test No.	Specimen	Anchor Length L_d (in)	Face Cover C_c (in)	Concrete Strength (ksi)	Average Bar Stress [2] (ksi)		Bond Factor $\frac{u}{\sqrt{f_c'}}$	Bond Strength Ratio [4]	Casting Position
					Meas-ured	Modi-fied[3]			
1	1-24-NS-B[1]	24	1	3.7	48.6		6.21	1.09	Bottom
12	1-24-NS-B	23.5	1-1/8	4.2	51.4	47.5	5.72		Bottom
3	1-16-NS-B[1]	16	1	3.7	40.5		7.76	0.64	Bottom
13	1-16-NS-B	16	1-1/8	2.5	54.1	51.7	12.12		Bottom
2	2-24-NS-T[1]	24	1	3.7	25.4		3.24	0.67	Top
11	2-24-NS-T	24	1-1/2	4.2	52.2	40.2	4.85		Top
4	2-16-NS-T[1]	16	1	3.7	39.9		7.64	0.81	Top
10	2-16-NS-T	16	1	2.6	40.8	40.8	9.47		Top

Note: [1] Tests on specimen #1 (without teflon sheet backing)
[2] The outer layer of bars for the two layer case
[3] Bar stress normalized to the cover thickness (1"), anchorage length (24")
e.g. for test 13: measured stress = 54.1ksi; Calculated bond factor = 11.54 for 1-1/8 in. cover; and 11.04 for 1 in. cover (Equation 2.9). Modified stress = $54.1 \times 11.04 \div 11.54$ = 51.7 ksi
[4] Bond strength ratio of the tests without teflon sheet to that with teflon sheet

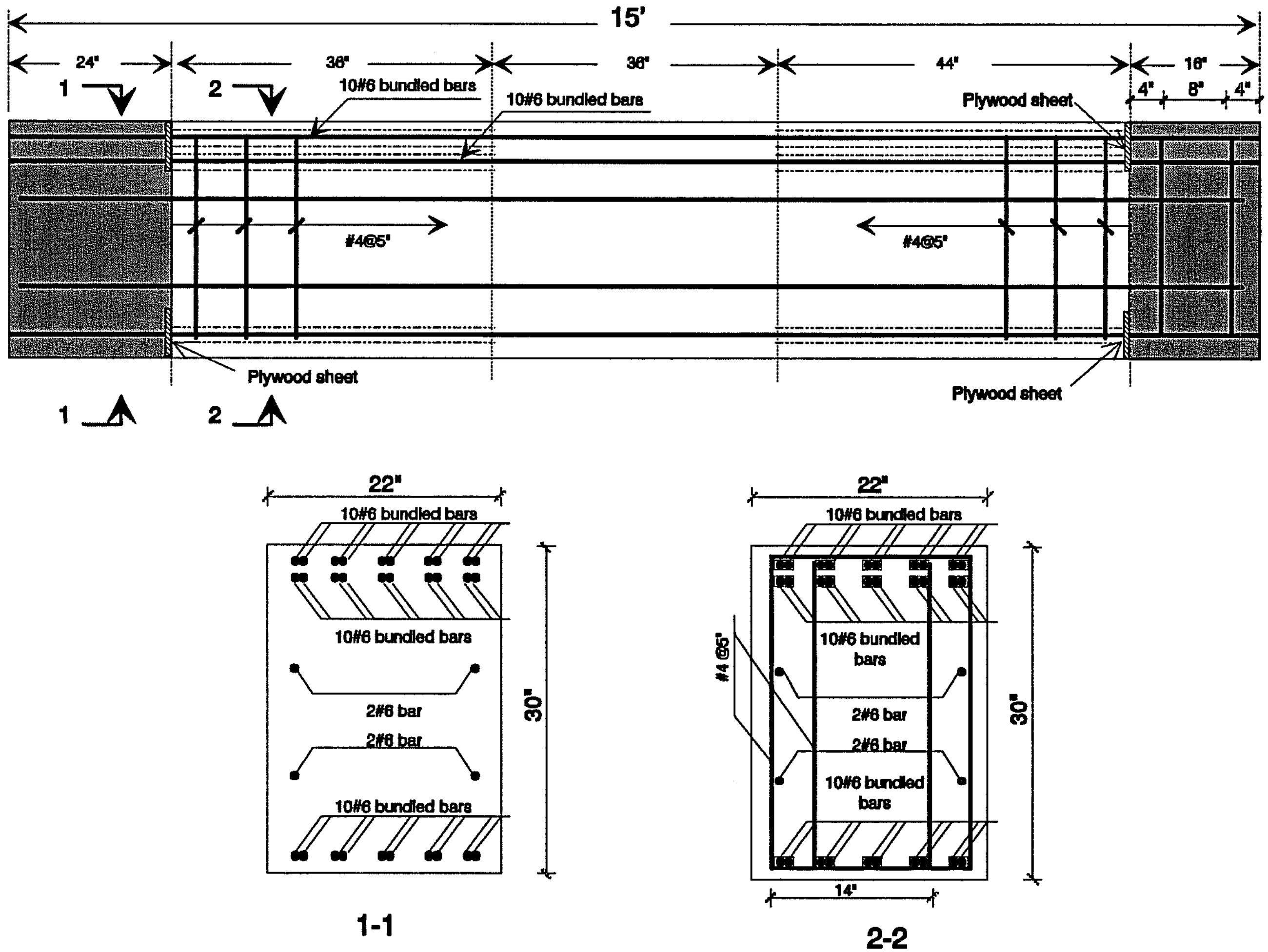


Figure 3.8 Details of specimen #1 (Plywood sheet backing)

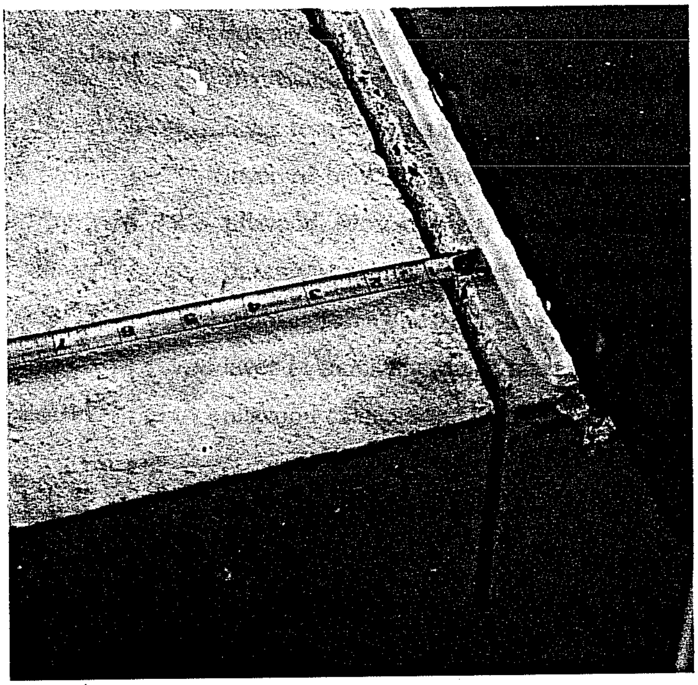


Figure ~~3.9~~ A.1 Gap in cover after removal of plywood

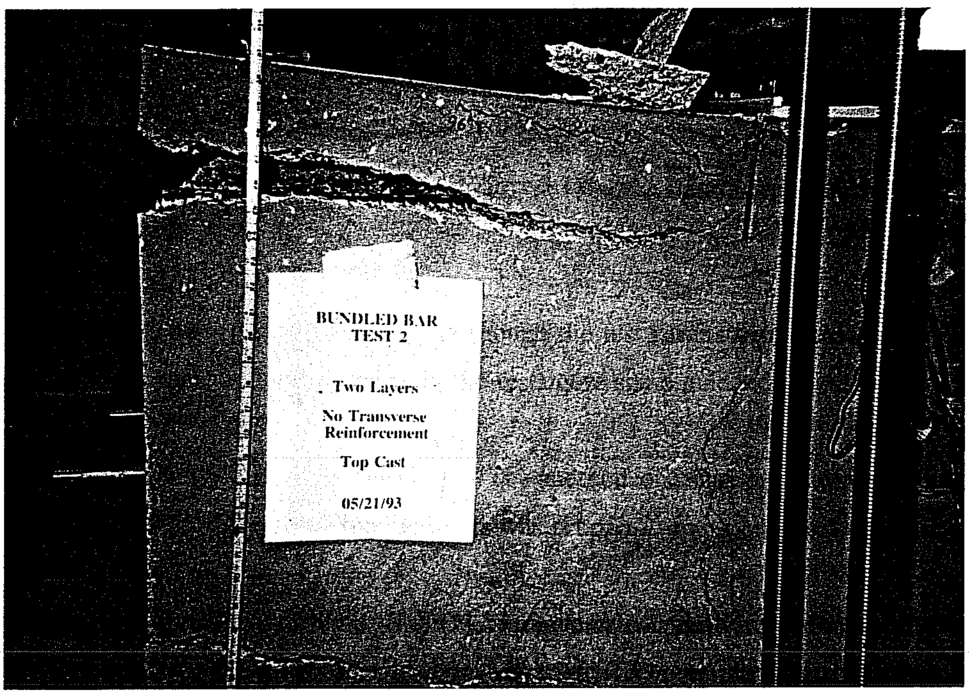


Figure ~~3.10~~ 4.2 Concrete cover rotation after bond failure strength. Table 3.2 shows that the measured bond strength from Specimen #1 was much lower than that from Specimen #2 (with teflon sheet backing). This phenomenon was more obvious in the two layer test than in the single layer test, because in the two layer test, the tension force in the outer layer of bars was not balanced by the bearing force from the concrete due to the soft layer of plywood. Most of the tension force in the outer layer of bars was transferred to the inner layer of bars as shown in Figure 3.11. This forced the concrete in the inner plane to resist not only the splitting force due to the tension from the inner layer of bars but also the shear stress from the outer layer of bars. The inner plane became a critical plane and usually failed prematurely. The use of plywood changed the mechanism of bond failure and all four tests had to be repeated.

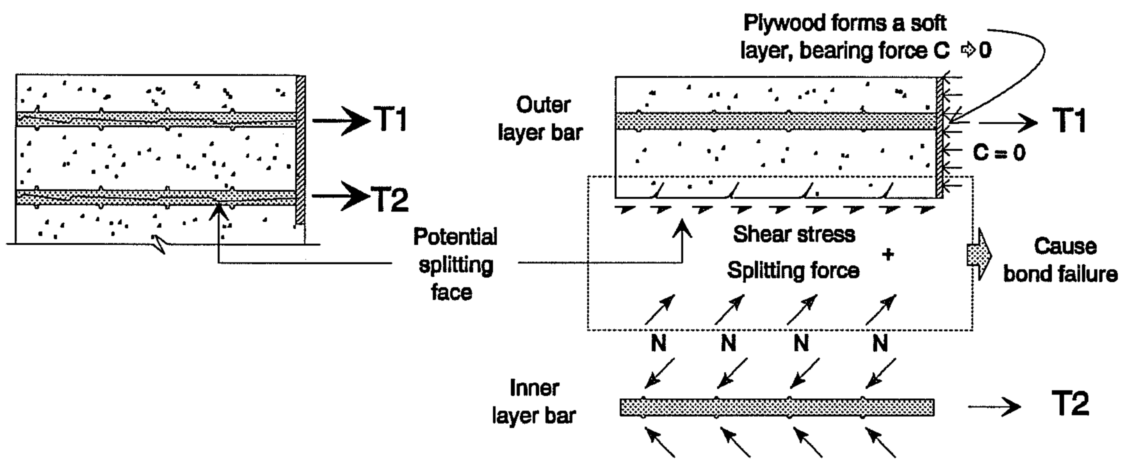


Figure 3.11 **Bond failure mechanism of test region (Specimen #1), Plywood was used as separating material**

Although the test results from this specimen did not reflect accurately the ultimate bond strength, the tests gave some guidance.

(1) The method of using partially unbonded bars to test the bond strength of multiple bundled bars was practical.

(2) The load calculated using the measured strains of the bundled bars was quite close to the load measured by the pressure transducer. Most of the strain gages survived the test and indicated that the strain gage data was reliable.

The accuracy of strain measurements is due in part to the location of the strain gages. The gages were placed about 3" from the intersection of test region and unbonded region, near the loading point. At this location, the beam had very little moment and thereby very little curvature, such that differential strain within the bars due to bending of the reinforcement was insignificant. As shown in Figure 3.12, if the curvature of the beam or shear deformation on the bars was large, there would be a large difference in the reading from a strain gage on the top of the bar and that from a strain gage at the bottom of the bar.

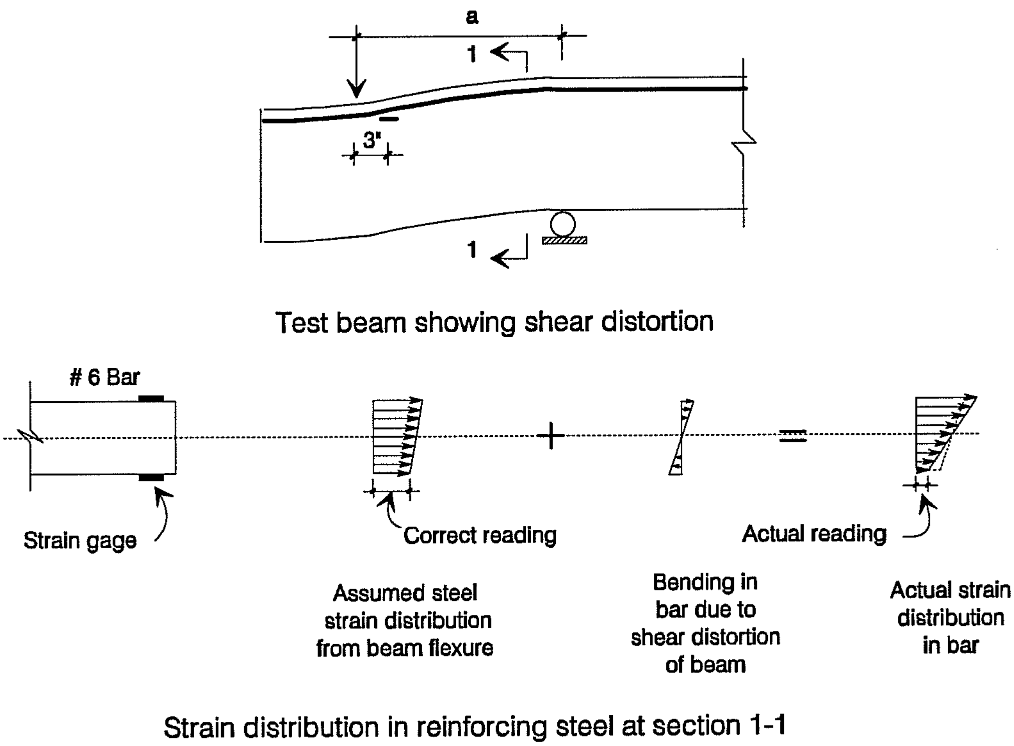


Figure 3.12 **Effect of shear deformation and curvature of beam on the reading of strain gage (Ref.5)**

(3) The measured strains along the cross section, shown in Figure 3.13 are nearly symmetric. Therefore, in the later tests, all the strain gages were placed on one half of the cross section. The data showed that the relative difference of the measured strain along the cross section was small. The strain distribution along the cross section was uniform.

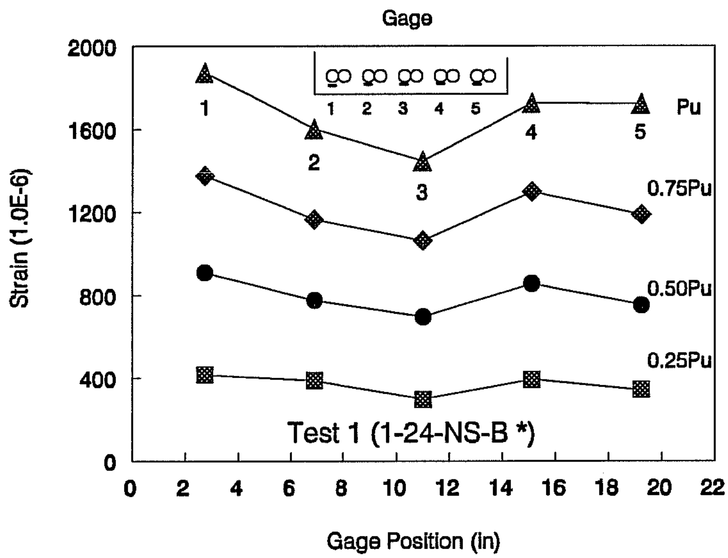


Figure 3.13 **Bar strain distribution across layer**

(4) The measured slip at the free end of the anchored bars was very small and inconsistent. The only significant slip occurred at failure. In the remaining tests, slip at the free end of the anchored bar was not measured.

All the failure modes were "face-and-side splitting". For the test without transverse reinforcement, longitudinal cracks always appeared first above the two corner bundles. After that, a third longitudinal crack sometimes appeared above the middle bundle (More obvious in the test of two layers of bars). At ultimate, splitting occurred through the plane of the bundled bars and the corner also separated from beam. The failure for the bundled bars without transverse reinforcement was sudden and brittle. At failure, the energy stored in the bundled bars was suddenly released and caused a second failure in the middle of the concrete block as shown in Figure 3.14.

For the test with transverse reinforcement, the crack patterns were different from those without transverse reinforcement. In addition to the two longitudinal cracks, which appeared above the two corner bundles, a third crack always appeared above the middle bundle. The middle bundle was not directly confined by a leg of the transverse reinforcement at this location. Transverse cracks appeared directly above the stirrups at later stages of the test. The transverse cracks appeared more

Figure ~~3.14~~ A.5 Second failure due to the shock at bond failure

obvious in the two layer test, in which the strain in the stirrups was higher than that in one layer. The failure mode was "face-and-side splitting". The failure occurred gradually and after considerable bar slip was observed.

After the test of one layer of bundled bars was completed, the beam was turned over, and the two layer tests were conducted. Since the bar at the bottom of the beam had already been tested, there was no reinforcement to restrain the end block rotation produced by the eccentric force from the two layers of bundled bars. A vertical crack formed in the concrete block as shown in Figure ~~3.15~~ 3.10. The rotation of the concrete block was detrimental to the bond strength of the bundled bars. In ~~later~~ subsequent specimens a layer of additional reinforcement was distributed below the single layer of bundled bars to control vertical cracking.

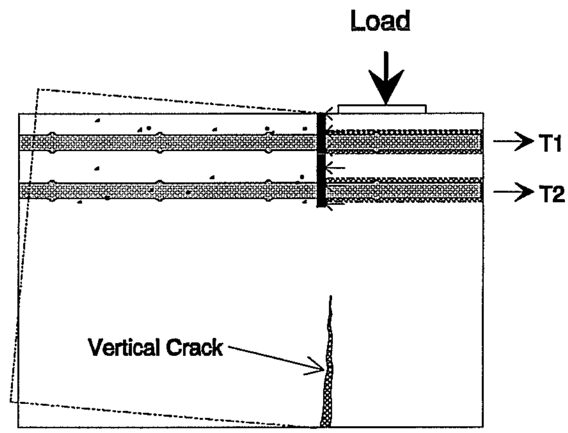


Figure 3.15 **Vertical cacking in beam due to eccentricity of loading on section**

3.3.3 Specimen #2 (without Shear)

The reinforcement configuration of Specimen #2 is identical to that of Specimen #1 except the position of the single layer and of two layers of bars were reversed. A thin teflon sheet backed by a piece of mild steel sheet metal was used to separate the test section as shown in Figure 3.16. The steel sheet kept the teflon flat. The smooth side of the teflon is contact with the concrete cover in the test region, allowed the concrete to split without confinement. Teflon is much stiffer than plywood and can sustain a great compression load without much deformation. The test region was not affected by the "separator" and teflon was used for all the remaining tests. To lessen the vertical cracks shown in Figure 3.15, an additional layer of reinforcement was placed under the one layer of bundled bars. The reinforcement details of Specimen #2 are shown in Figure 3.17.

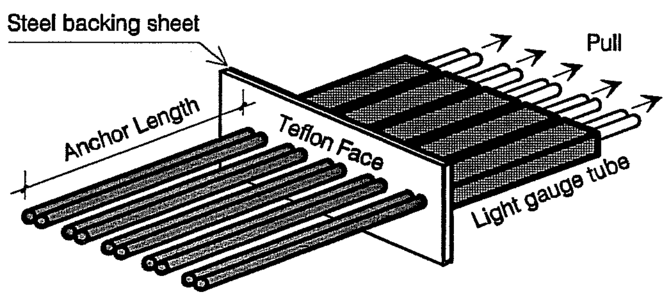


Figure 3.16 **Teflon sheet backing system**

Specimen #2 was used for eight tests by removing and recasting the test regions after failure. Most conclusions in this paper are based on the tests of this beam.

3.3.4 Specimen #3 (with Shear)

Specimen #3 was designed to determine bond strength in the presence of shear. In this beam, one layer of bars was at the top of the beam and the two layers were on the bottom. A teflon sheet was placed at either end of the test region. The detail of specimen #3 is shown in Figure 3.18. In this specimen, the test region was between the loading point and the support (Figure 3.3). At the ends of the beam, the bundled bars (1 ft. length), directly under the loading point, were unbonded. This arrangement appeared to be the only way to introduce shear within the test region and without the load restraining splitting at the same time. To provide moment capacity at the end of the beam to transfer load to the beam, 6 #6 rebars were placed as shown in Figure 3.18.

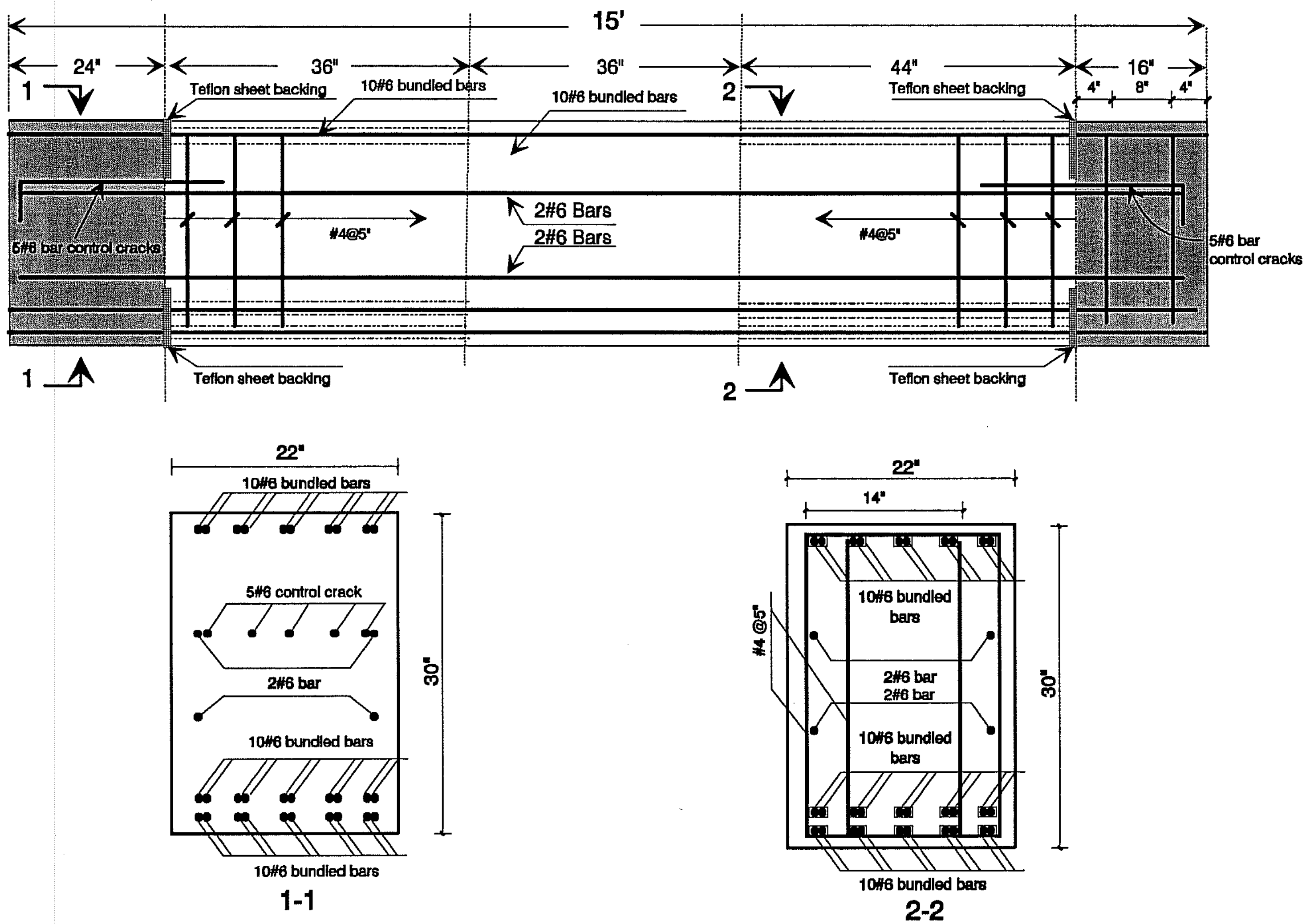


Figure 3.17 Details of specimen # 2 (with teflon sheet backing)

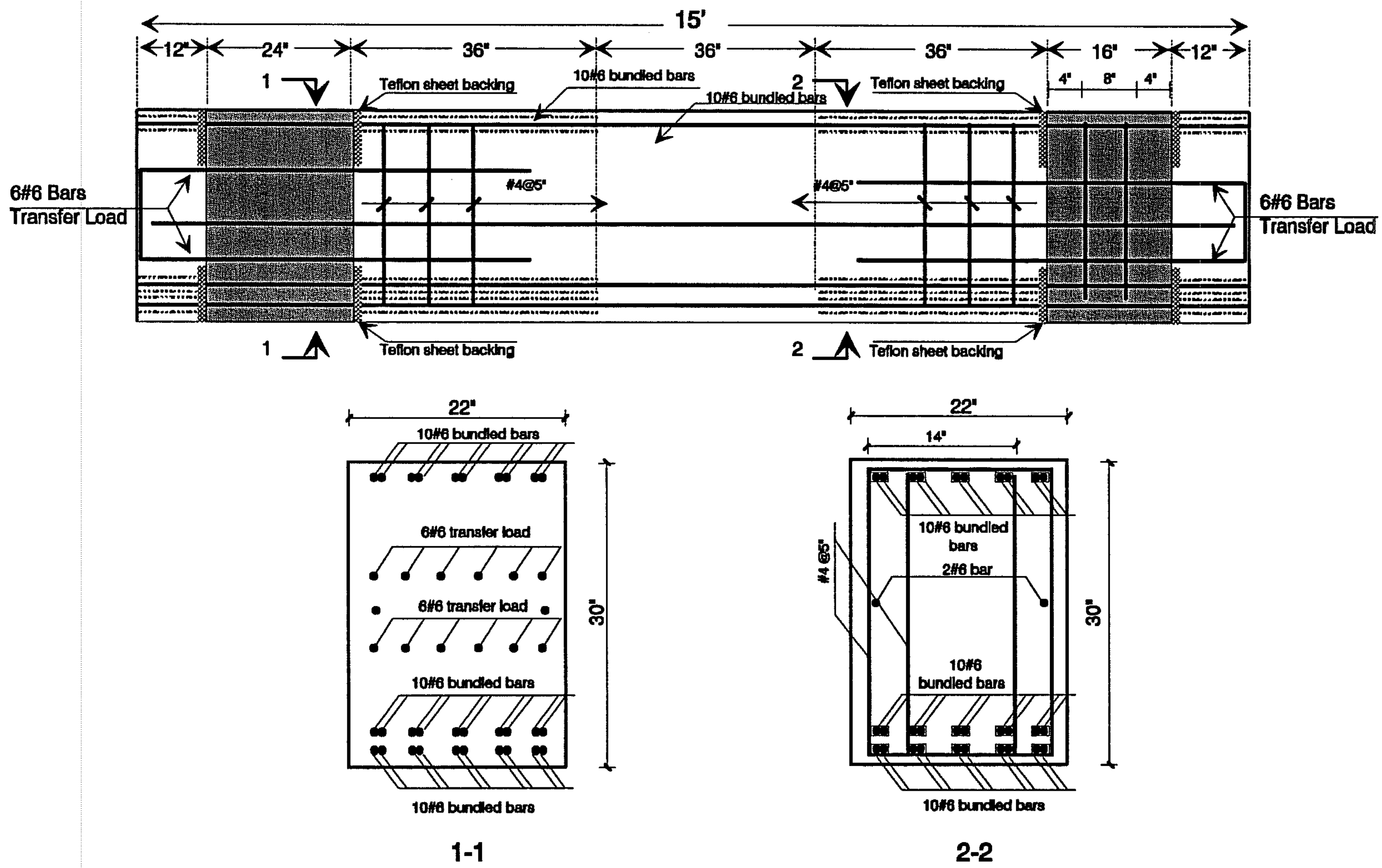


Figure 3.18 **Details of specimen #3 (with teflon sheet, design for shear)**

3.4 Materials

3.4.1 Concrete

The concrete strength was based on typical strength used by TxDOT. The nominal compressive strength at 28-days was 24 MPa (3500 psi). The actual compressive strength from standard 6"×12" cylinders ranged from 17 MPa (2500 psi) to 28 MPa (4100 psi). Because of the tight spacing of reinforcement, 3/8" coarse aggregate was used and the slump of the concrete at placement was 6 to 8" to ensure good consolidation of concrete. Usually, when the ready mix truck arrived, the concrete had a slump less than 6". Before concrete was placed, water was added to produce the specified slump. The concrete was placed in three to four lifts. Each layer was consolidated with hand held vibrators.

Based on the cylinder strength at 7 days, 14 days, 28 days and on the day of last testing, a strength curve was developed. The concrete strength at the day of testing was determined from this curve. A typical concrete strength curve is shown in Figure 3.19. For each specimen, split cylinder tests were conducted to measure the concrete tensile strength.

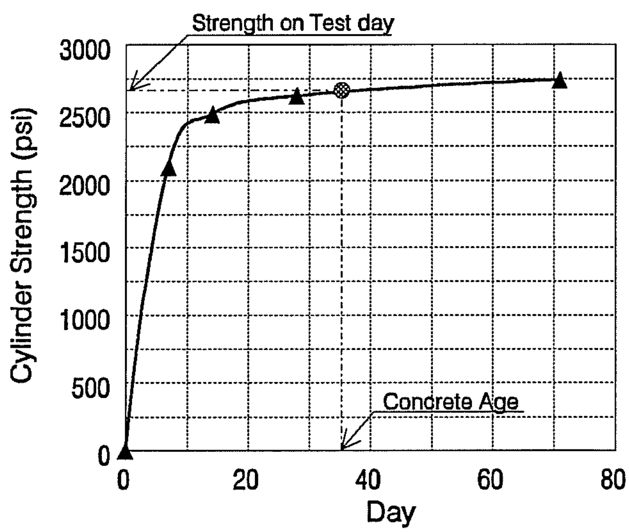


Figure 3-19 **Typical concrete strength-age curve**

3.4.2 Reinforcement

All longitudinal reinforcement was from the same heat. The longitudinal reinforcement was #6 bars and the transverse reinforcement was #4 bars. The nominal yield strength was 414 MPa (60 ksi). After all the tests were finished, 4 longitudinal bars and 3 transverse bars were removed from the specimen and tested. The yield stress was 422 MPa (61.2 ksi) for #6 bars and 456 MPa (66.1 ksi) for the #4 stirrups.

3.5 Specimen Construction

After strain gages were bonded to the reinforcement, light gage steel tubes were placed over the two-bar bundle in regions where the bars were to be unbonded. Strain gages were covered by the tubes and gage wires were threaded through holes in the tubes. Finally, silicon caulk was inserted to seal the gaps between bundles and the ends of the tubes to prevent cement paste from getting into the tubes.

The bars were fixed in place while the cage was tied. First, the stirrups were placed on the cage, but left untied. Plywood guides were placed around the bars at either end of the cage to fix the bars in proper pattern. After that, all bundled bars and stirrups were tied. Diagonal bars were tied in the middle of the beam to help stabilize the cage. Figure 3.20 shows a picture of the cage. The separators (plywood, or teflon) were positioned in the beam and the whole cage was carefully lifted and placed in the form.

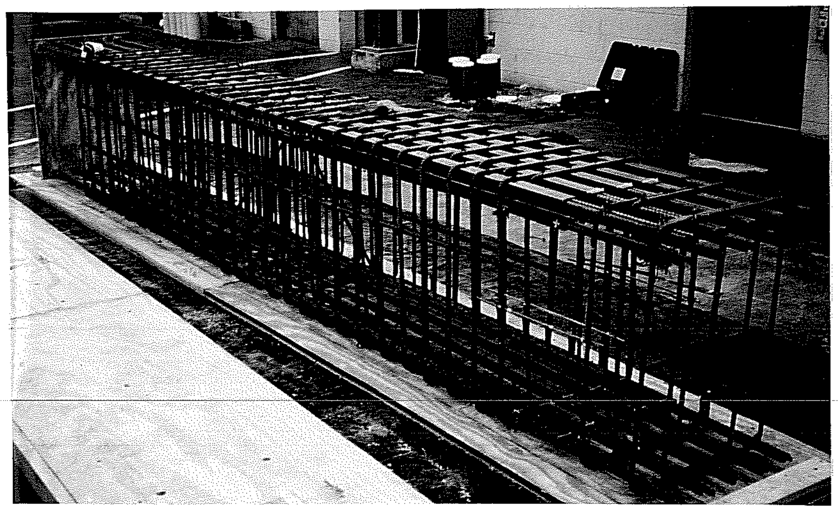


Figure 3.20 Reinforcement cage of specimen

3.6 Loading System

The test setup for the beam specimen was very simple. Details of the test set-up are shown in Figures 3.21 and 3.22. The end of specimen with test bars was loaded as a cantilever beam. At the loading end, two 100 ton hydraulic rams were set on top of a loading beam. The rams were connected to the same pump so that the same pressure was maintained in the two rams. The load was applied to the specimen by a roller which allowed the beam end to rotate freely. Each of the hydraulic rams were anchored to the test floor by four rods. At the opposite end of the beam, another loading beam was also tied to the test floor by eight rods. Between the two loading beams was a roller supporting the beam. The floor support was at the intersection of the unbonded region and the middle bonded region of the beam. When load was applied, a crack appeared at the top of the beam over the support, and tension was introduced in the bundled bars. Since the load at failure exceeded 130 kips, the self-weight of specimen, rams and loading beam were neglected.

3.7 Testing Procedure

The support roller was fixed to the floor using a thin layer of quick-setting grout to even the bearing surface. After the test region and the loading point were marked on the specimen, the specimen was moved in position. A loading plate was placed on the specimen with grout between the concrete and plate to ensure an even bearing surface. Then the loading beams, rams and plates were positioned. The loading apparatus were carefully aligned and the loading bars were tightened. A picture of the test set-up is shown in Figure 3.23.

Before testing to failure, the specimen was subjected to low loads and unloaded several times. The pre-loading permitted a check to examine that the loading system, strain gages, linear pots and other instruments were working properly.

Generally, the test procedure was the same for all the tests. The load was increased at 5-10 kips per step until the first longitudinal crack appeared in the test region. After that, the load was

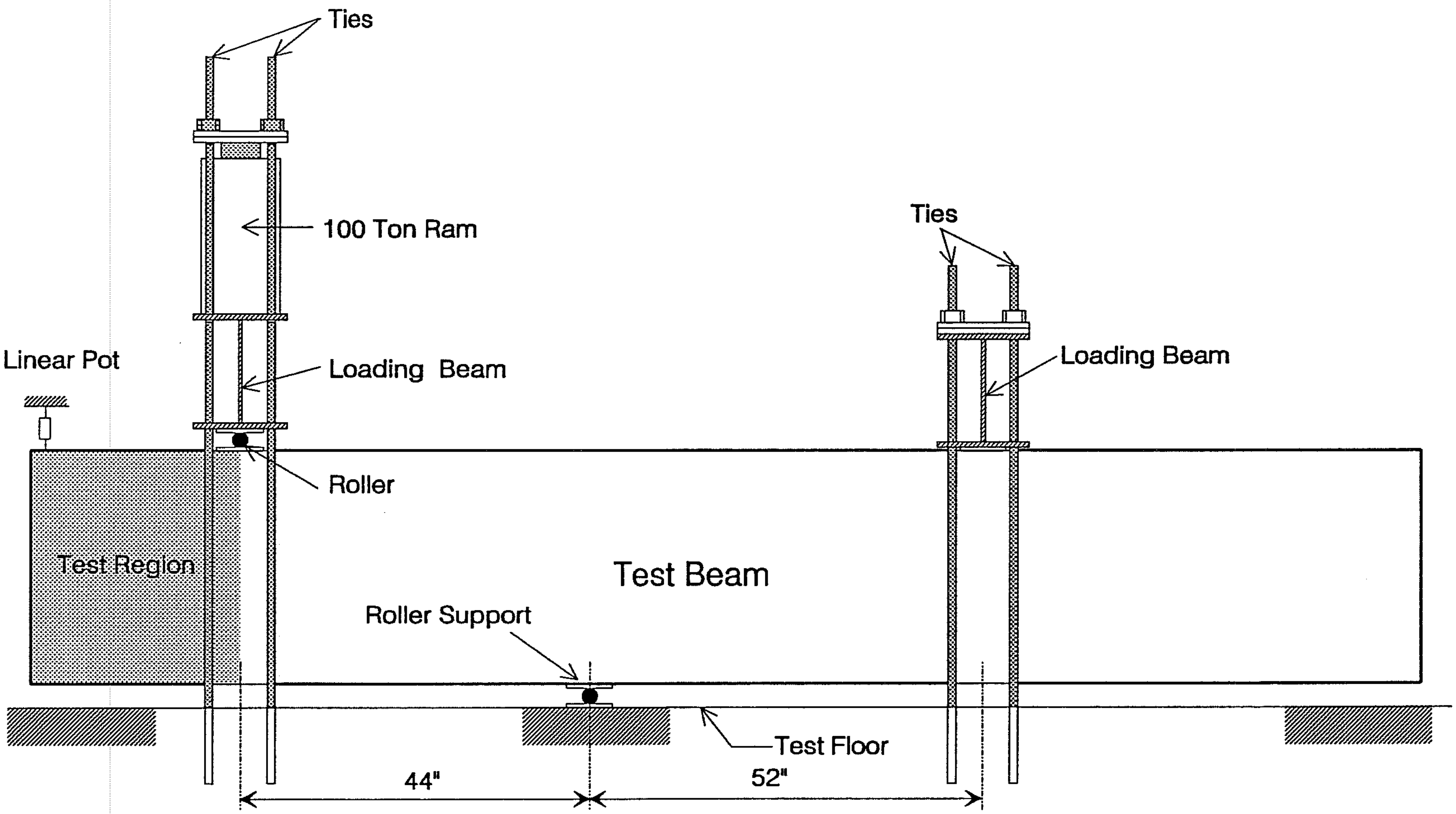


Figure 3.21: Side view of test setup

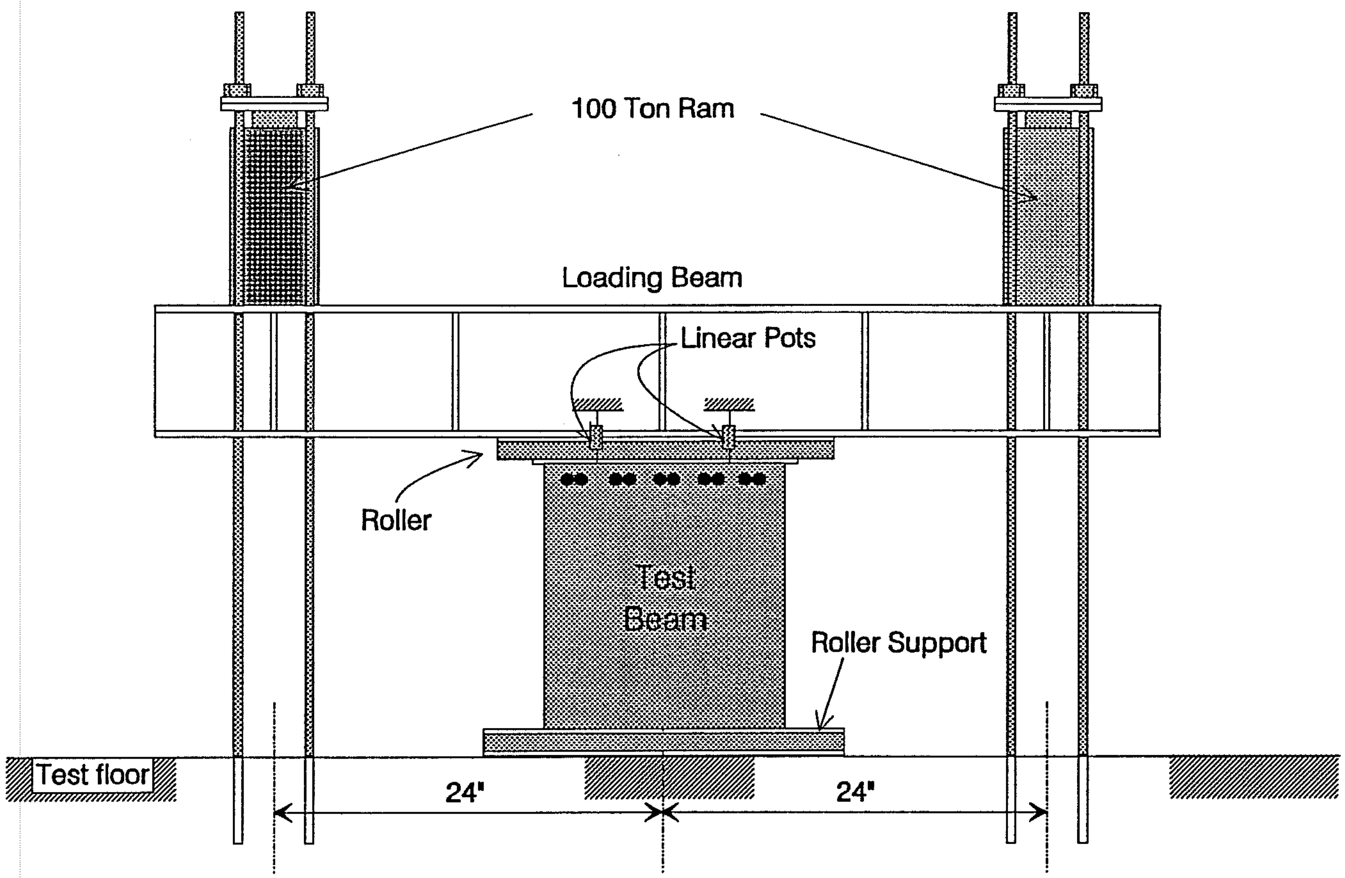


Figure 3.22 **Test setup front view**

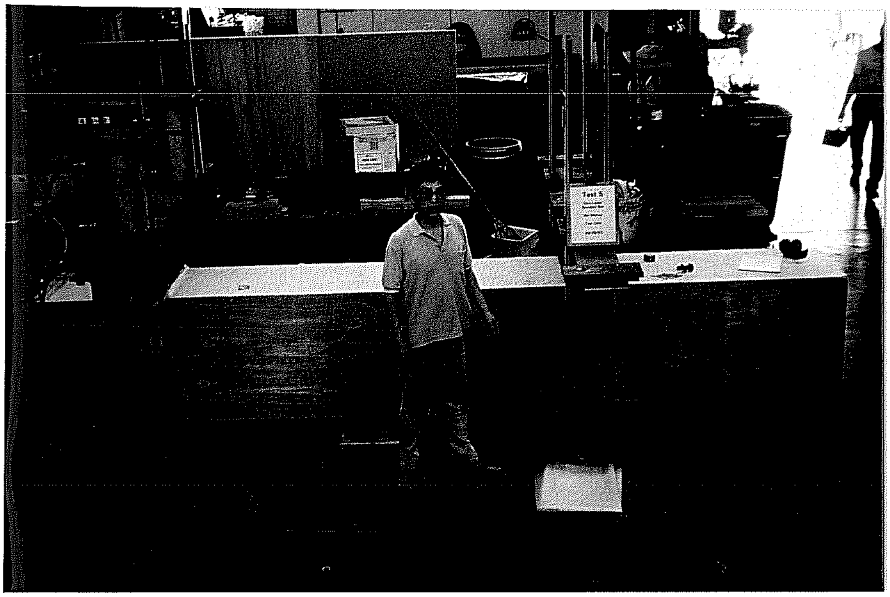


Figure 3.23 Test setup

increased by 2 to 3 kips per step, which limited the increase of stress in the bars to about 1 ksi per step, until bond failure occurred. The load was applied manually by a hydraulic pump. The pressure was measured by an electronic pressure transducer and monitored by a mechanical pressure gage. Deformation at the end of the beam and load were recorded and plotted by an X-Y recorder. The load-deflection curve was used as a visual indication of the progress of the test. All data were recorded by a data acquisition system. The time interval between load steps depended on the time required for the system to scan and print the data. This usually took less than 1 minute. Periodically, the test region was carefully inspected to see if there were any cracks or how the cracks were developing. The cracks were marked and the corresponding load stage was indicated beside them.

Test ended when either the concrete cover on the test region suddenly split or the beam would take no more load. Upon completion of the test, photographs were taken to record crack patterns on both sides and on the top of the test region. The concrete cover was removed and failure plane was examined. The thickness of the cover and the clear spacing of bundles were measured again. The single layer test was always performed first, then the beam was turned over, and the two layer case was tested.

3.8 Strain and Deflection Instrumentation

Stress in the reinforcement was calculated using strains recorded by the strain gages on the reinforcement. In specimen #1, all the bundles were gaged, but there was only one strain gage per bundle. In specimens #2 and #3, three of the five bundles were instrumented with each bar within a bundle having a strain gage on it. The strain gages on the stirrups were placed just below the probable splitting plane. At these locations, the strain gages would most likely reflect the true strain of the stirrup. Gages located some distance from cracks may not be accurate because the bar stress is transferred rapidly to the concrete away from crack. Figures 3.5, 3.6, and 3.7 show the positions and arrangement of the strain gages on the specimen.

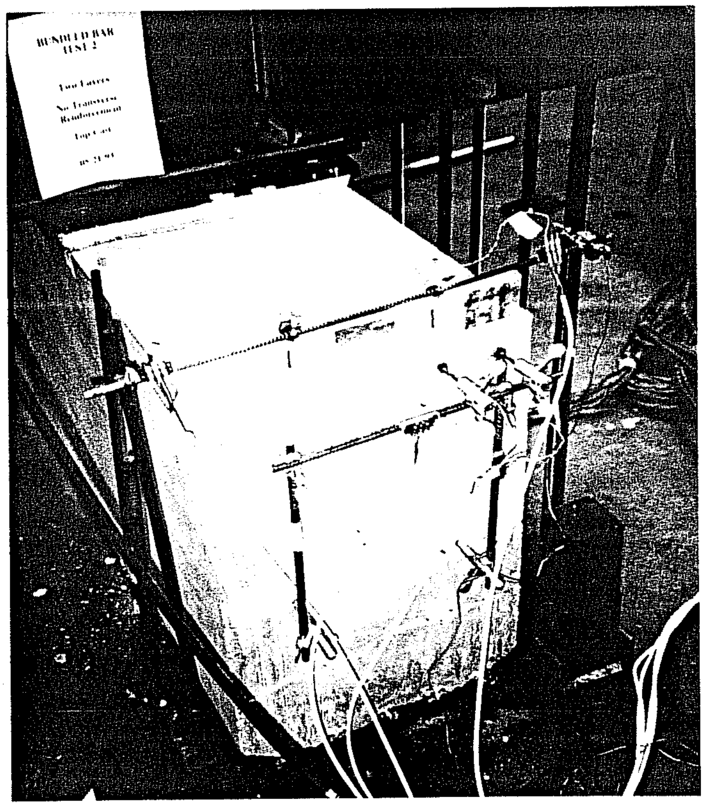


Figure 3.24 **Position of linear pots measuring beam end deflection and slip at the free end of anchor bars**

Two deformation transducers were placed at the end of the beam to measure the deflection and any twisting of the beam during the test. In the test of specimen #1, the slip of the free end of anchored bars was measured as shown in Figure 3.24. However the data showed that the slip of the free end before the bond failure was so small that it was beyond the accuracy of the testing method. Therefore, slip was not measured in later tests.

CHAPTER 4
EXPERIMENTAL RESULTS

4.1 Introduction

Load-stress relationships for twelve tests are presented. The behavior of the bundled bar is described in terms of the failure mode and crack patterns in the test region. For clarity, some keyterms used in the discussion are explained in Figure 4.1.

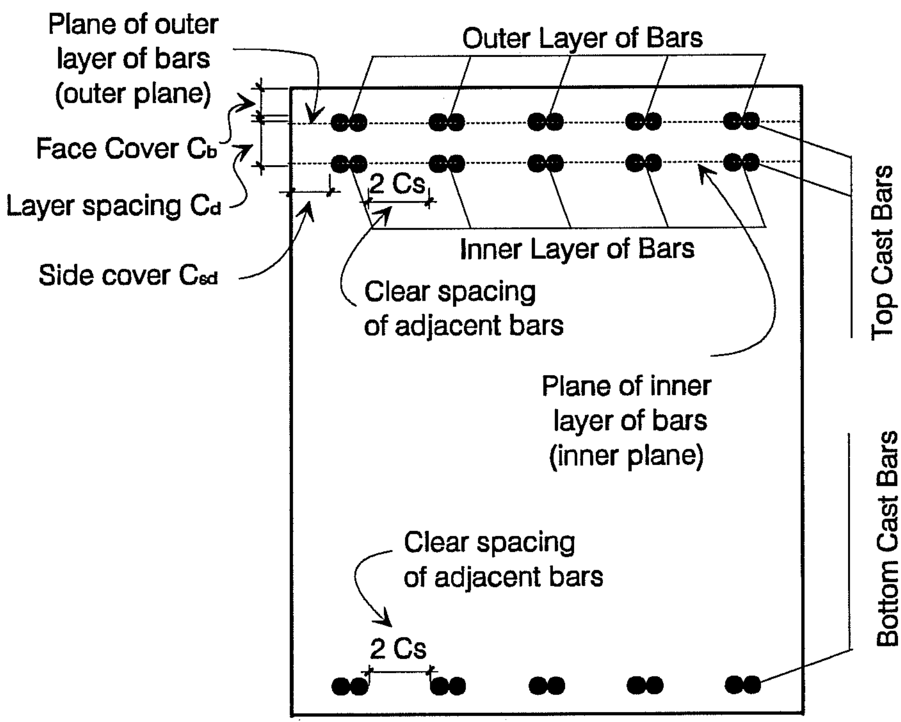


Figure 4.1 Keyterms

4.2 Tests without Shear

In eight tests, bundled bars were subjected to tension without shear acting on the bar test region. The data from these eight tests were used to compare the bond behavior of two-bar bundles in one and two layers, cast in top and bottom positions, reinforced with and without transverse reinforcement.

4.2.1 Bar Stresses within a Bundle

A typical load-stress relationship for individual bars is shown in Figure 4.2. The stresses of the two bars in a bundle generally were close. After failure, the concrete cover was removed. The splitting plane and cracking around the bundled bars were examined. At failure, both bars in a bundle always slipped the same amount and there was no observable relative movement between the two bars. In evaluating the data, stresses of bundled bars are taken as the average measured stresses of two bars in a bundle.

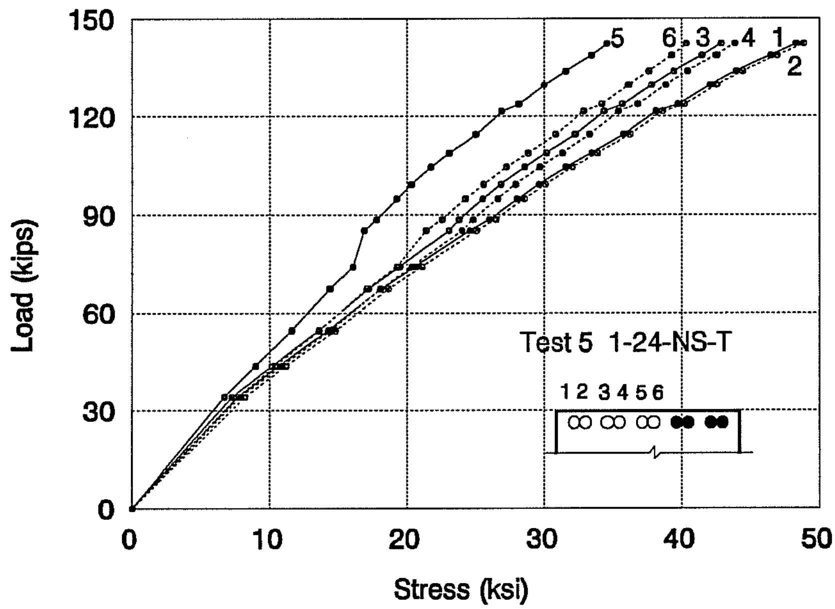


Figure 4.2 Load-stress response of individual bars

4.2.2 One Layer

When the stresses in the bundled bars reached 90 percent of the peak stress, longitudinal cracks appeared over two corner bundles. The cracks lengthened as the load increased. When the cracks were about two thirds of the anchorage length, the bars failed in bond. The failure was a "side-and-face splitting" mode. The bond failure of bars without stirrups was brittle and quite violent as the energy was released from the bundled bars. A typical failure mode is shown in Figure 4.3.

In the tests of bundled bars with stirrups, the transverse reinforcement confined the bars (crossed the splitting plane) and longitudinal cracks above the two corner bundles appeared very late in the test. In one test, cracks were visible only after failure. A third longitudinal crack appeared above the middle bundle since there was less confinement from the stirrups at this location. The measured stress in the transverse reinforcement was around 34.5 MPa (5 ksi) at the peak load. The low stress correlated with the absence of transverse cracks. The failure of bundled bars with stirrups was less severe than that without stirrups. The failure mode could be termed a "side-and-face split" failure as shown in Figure 4.4.

The load-stress relationship for four one layer tests is shown in Figures 4.5 through 4.8. In Figure 4.7, the stirrup stress is also included and shows that the stress in transverse reinforcement is low even at failure. Along with the curves, the measured loads and stresses near the peak load are included for easy comparison.

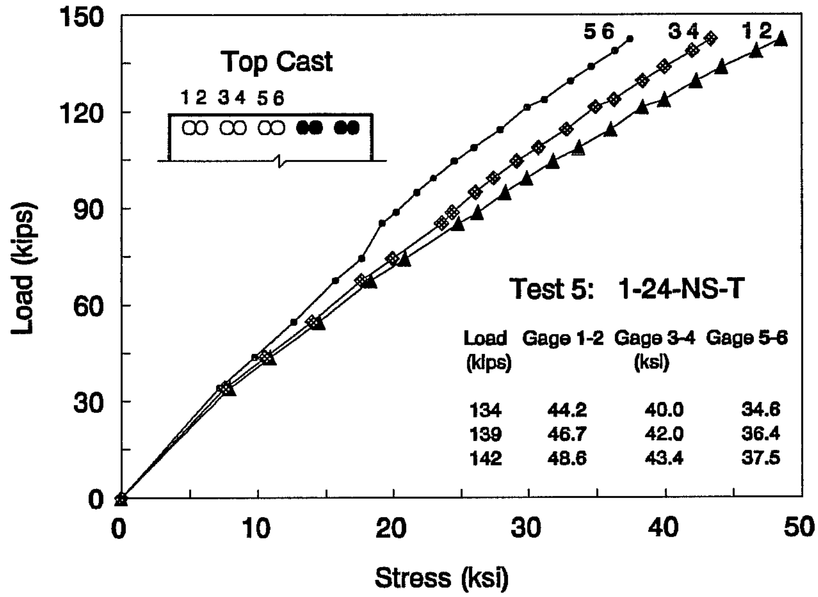
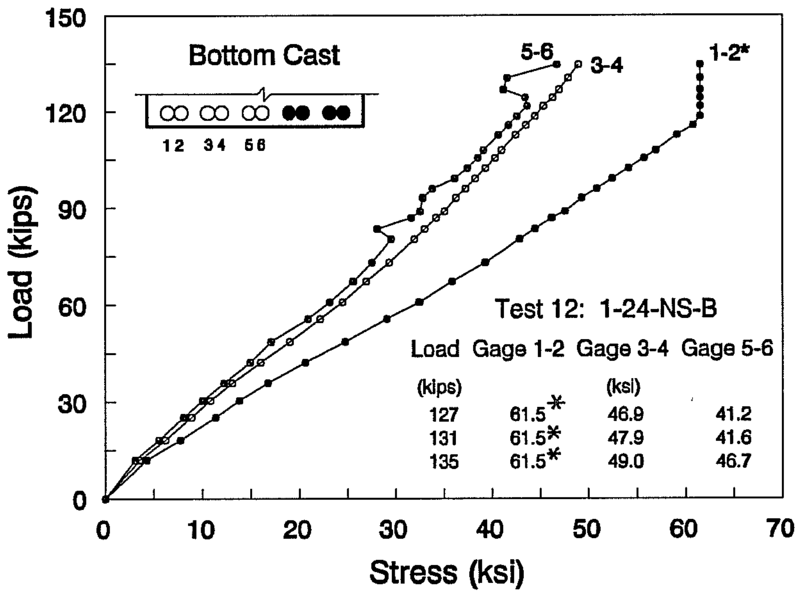


Figure 4.5 **Load versus stress of bundled bars (Test 5: 1-24-NS-T)**



* Stress of this bundled bars seem anomolously large

* The calculated load does not consist with the measured load

Figure 4.6 **Load versus stress of bundled Bars (Test 12: 1-24-NS-B)**

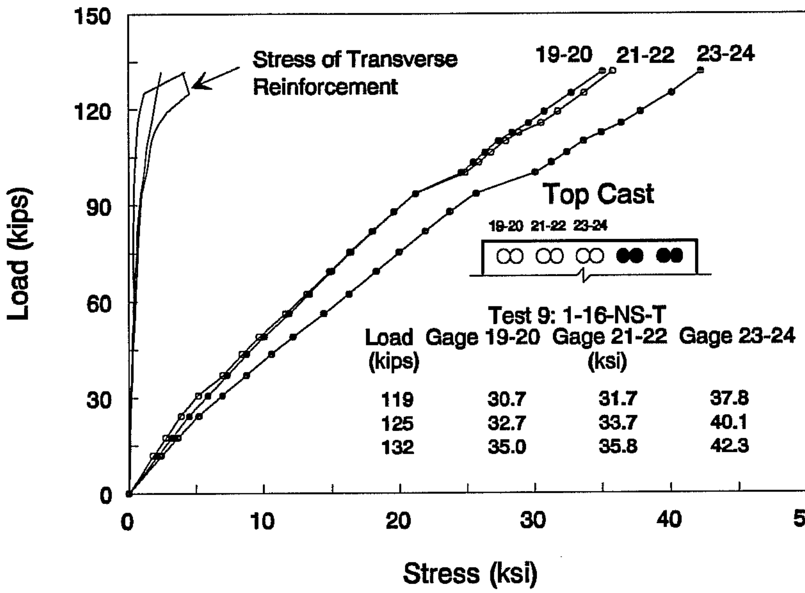


Figure 4.7 **Load versus stress of bundled bars (Test 9: 1-16-NS-T)**

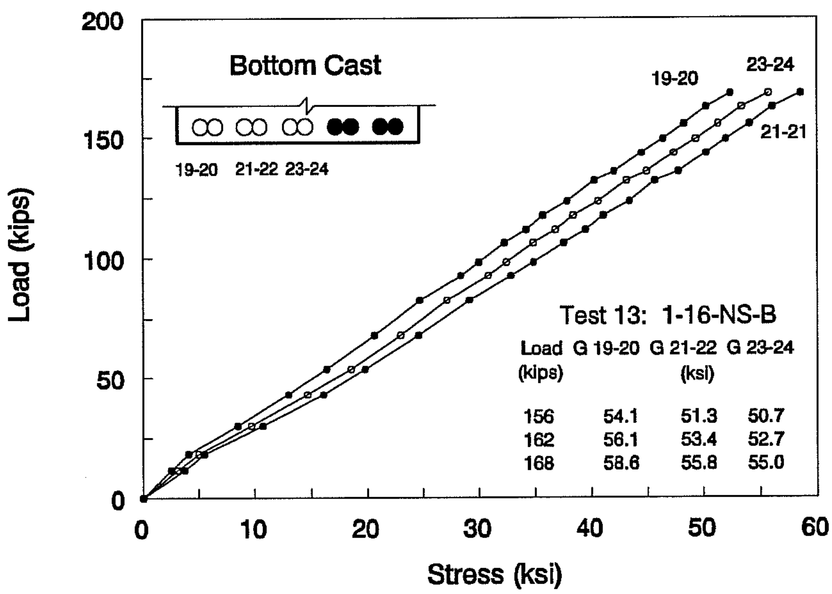


Figure 4.8 **Load versus stress of bundled bars (Test 13: 1-16-NS-B)**

4.2.3 Two Layers

Due to the large eccentric force applied to the beam end section from the two layers of bundled bars, there were still some vertical cracks at the bottom of the beam as shown in Figure 3.15, even though the plywood sheet was replaced by a teflon sheet and additional reinforcement was placed to control this cracking. However, the cracks were much smaller than those on specimen #1. The vertical cracks stopped below the test region and the rotation of the concrete block was also small. The vertical cracks were not considered to affect the test results.

Two longitudinal cracks appeared above the two corner bundles and the third crack appeared above the middle bundle. As the load increased, the longitudinal cracks lengthened and bars failed suddenly when the cracks reached about two thirds of the anchorage length. Before failure, horizontal cracks along the plane of bundled bars could be seen at the free end of the anchored bars. Usually the crack appeared first in the plane of inner bars. If the load was maintained or increased a little, a second crack formed in the plane of outer bars just before the failure. The bond failure of bars without stirrups was brittle and sudden. As shown in Figure 4.9, the side concrete cover spalled off. The figure also shows that splitting was through both the outer and inner planes.

In the tests of two layers of bundled bars with stirrups, longitudinal cracks appeared first above the two corner bundles, then were followed by a third longitudinal crack above the middle bundle. At about ninety percent of the peak load, transverse cracks appeared in the cover directly above the stirrups. The failures were "side-and-face split" modes. The same sequence of cracking occurred as in the tests without stirrups. At the free end of the anchored bars, the horizontal crack appeared first in the plane of inner bars. Then a second crack appeared in the outer plane of bars. As soon as the crack appeared in the plane of outer bars, the specimen failed. However, with the stirrups, the failure process was more gradual and the crack in the plane of outer bars was more obvious than in the tests without stirrups. The typical failure mode is shown in Figure 4.10.

The load versus stress relationship is shown in Figures 4.11 through 4.14. Stirrup stresses are also included in Figure 4.13. As shown in this figure, the stress in stirrups in the two layer test was 128 MPa (20 ksi) at the peak load, which was more than twice that in one layer test.

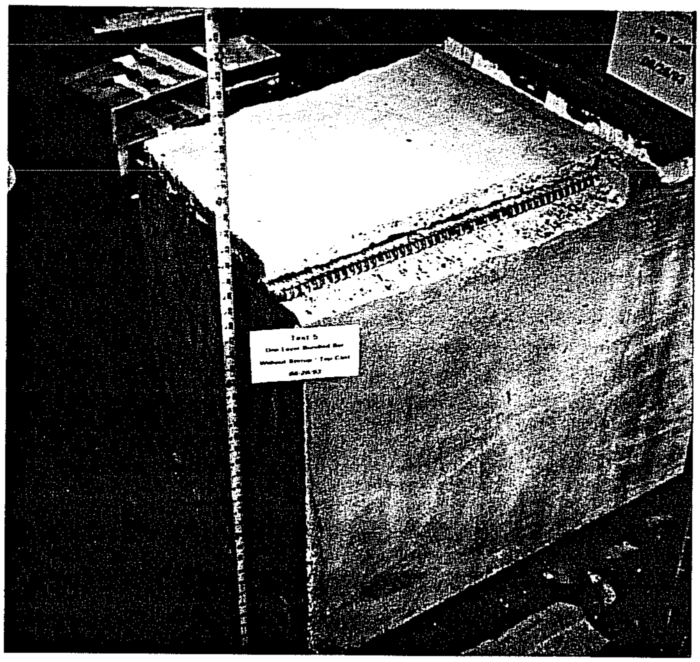


Figure 4.3 **Failure mode for one layer of bundled bars without transverse reinforcement (Test 5 1-24-NS-T)**

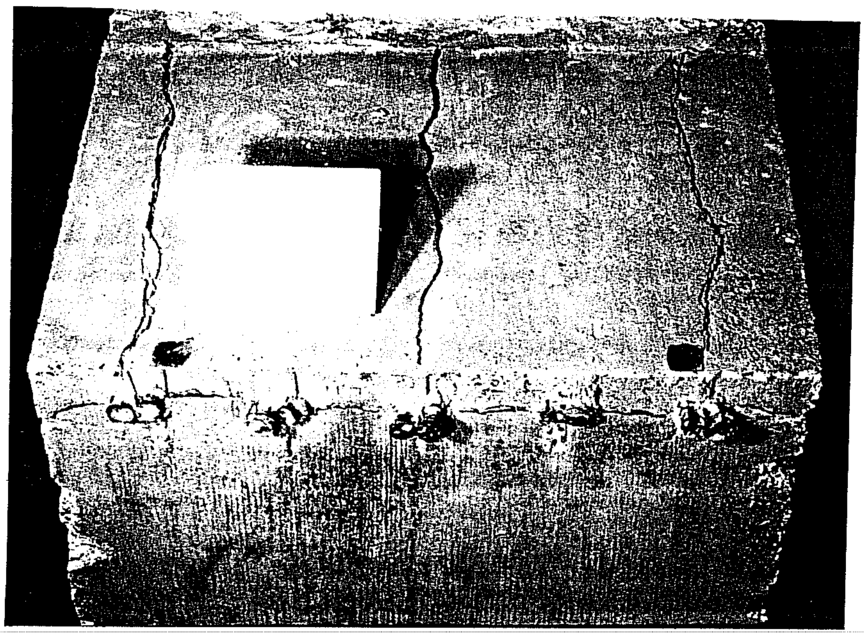


Figure 4.4 **Failure mode for one layer of bundled bars with transverse reinforcement (Test 13 1-16-NS-B)**

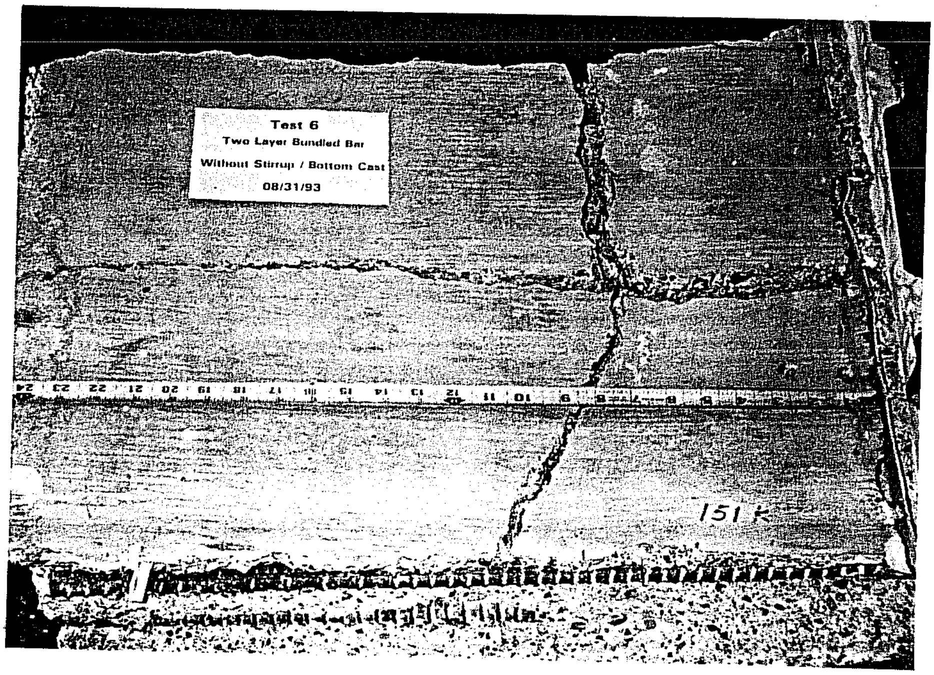


Figure 4.9 **Failure mode of two layers of bundled bars without transverse reinforcement (Test 6 2-24-NS-B)**

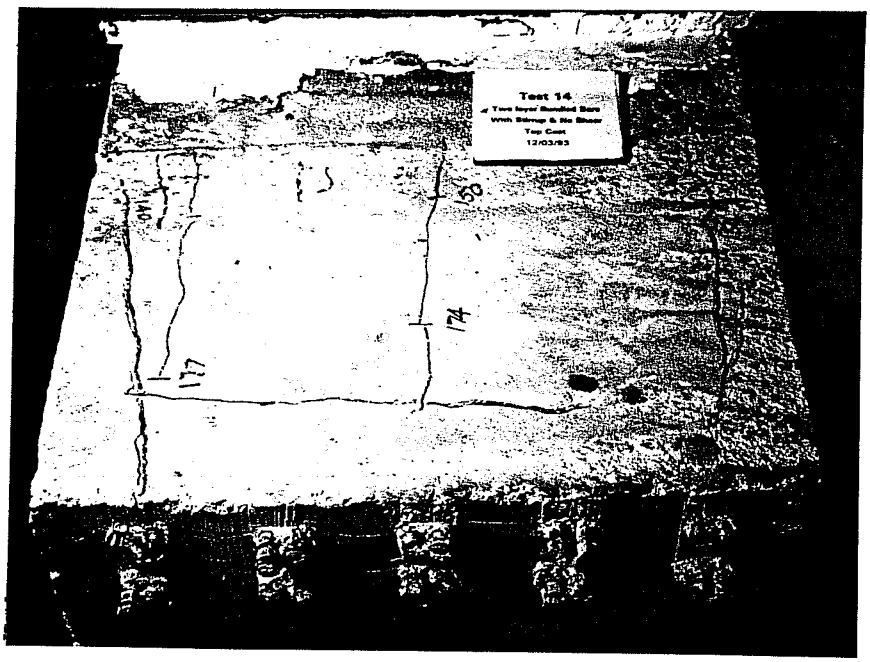
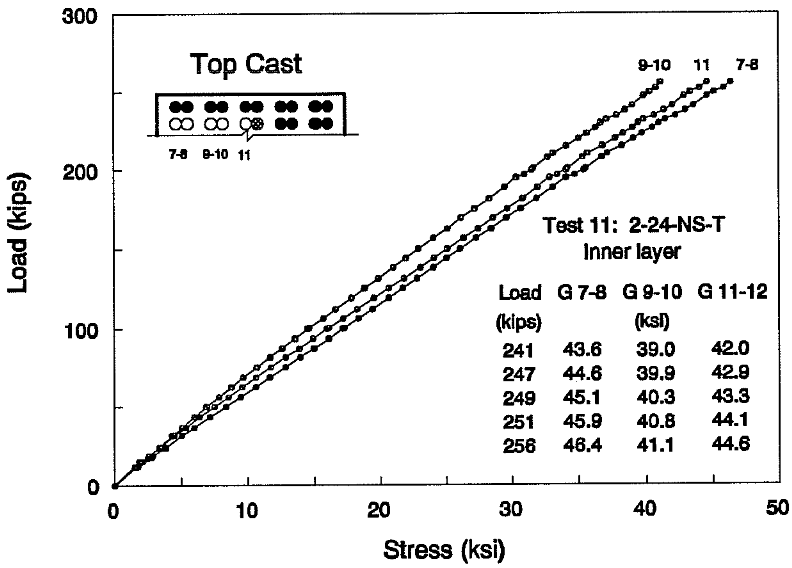


Figure 4.10 **Failure mode for two layers of bundled bars with transverse reinforcement (Test 14 2-16-NS-T)**



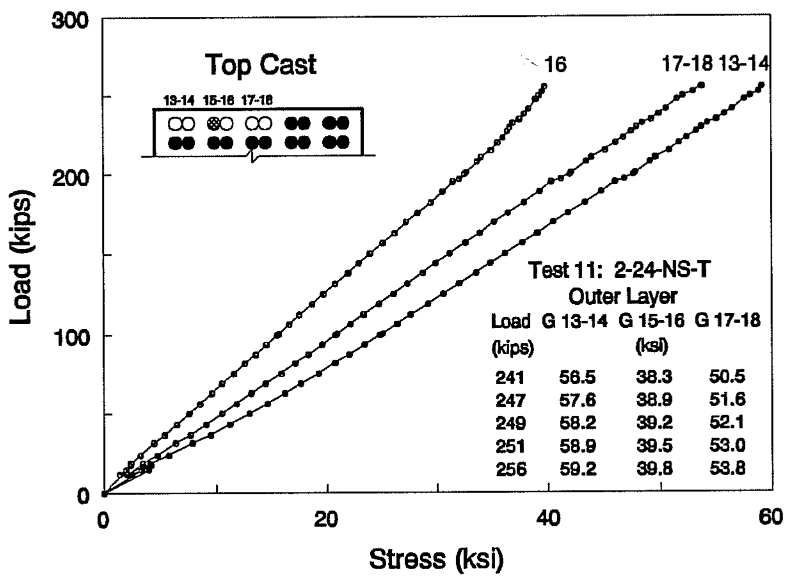
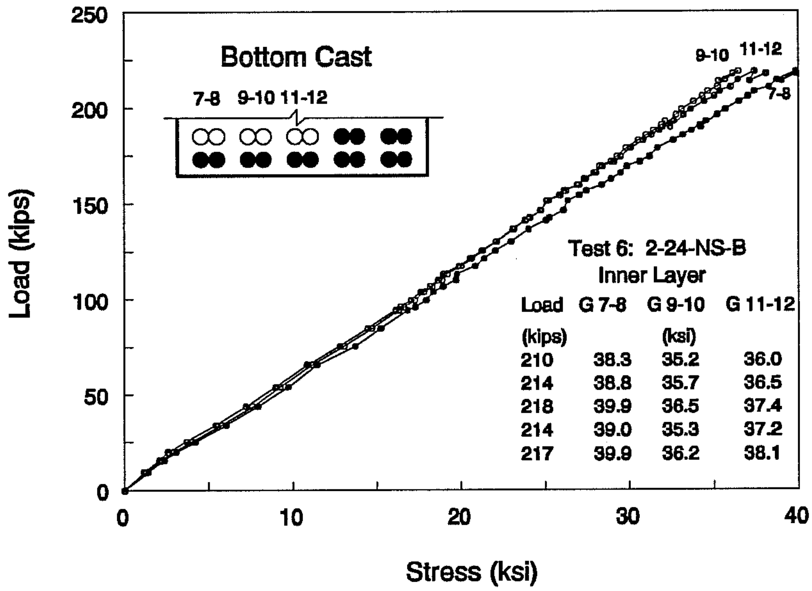


Figure 4.11 Load versus stress of bundled bars
(Test 11: 2-24-NS-T)



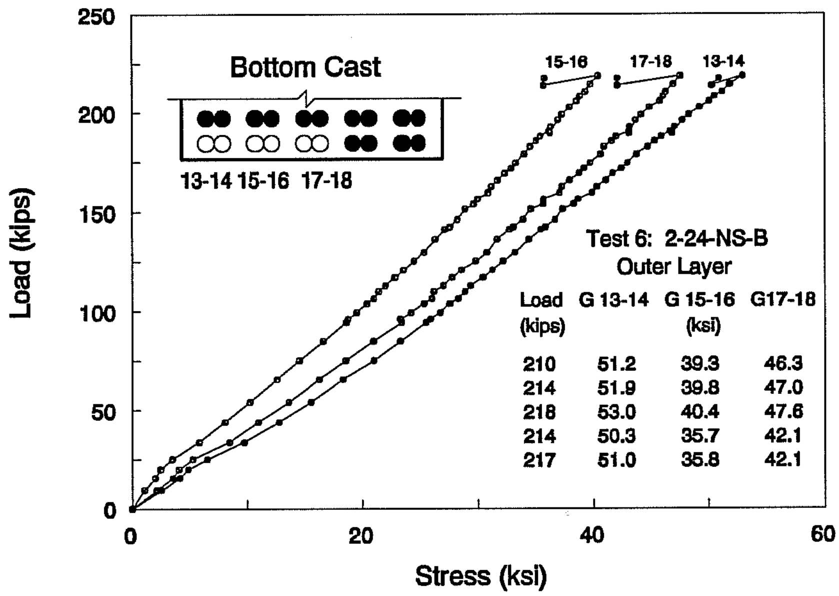
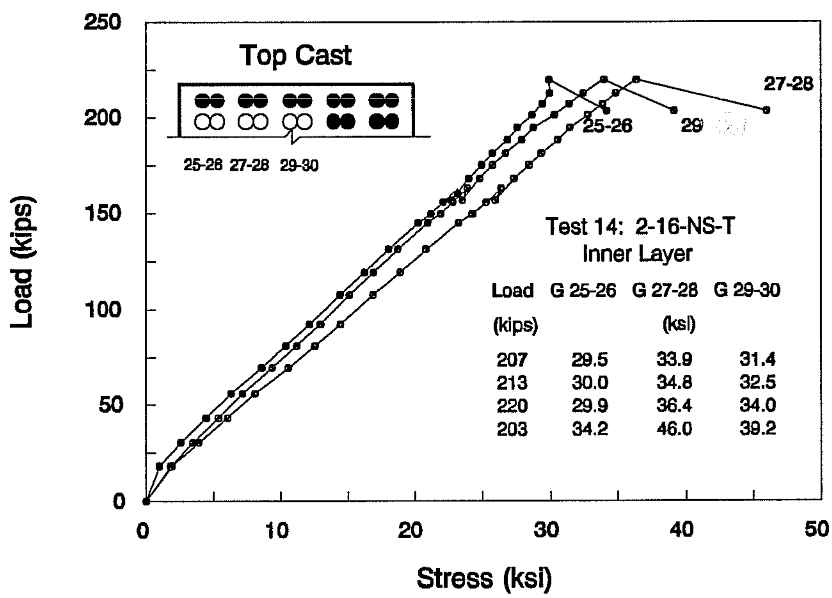


Figure 4.12 Load versus stress of bundled bars (Test 6: 2-24-NS-B)



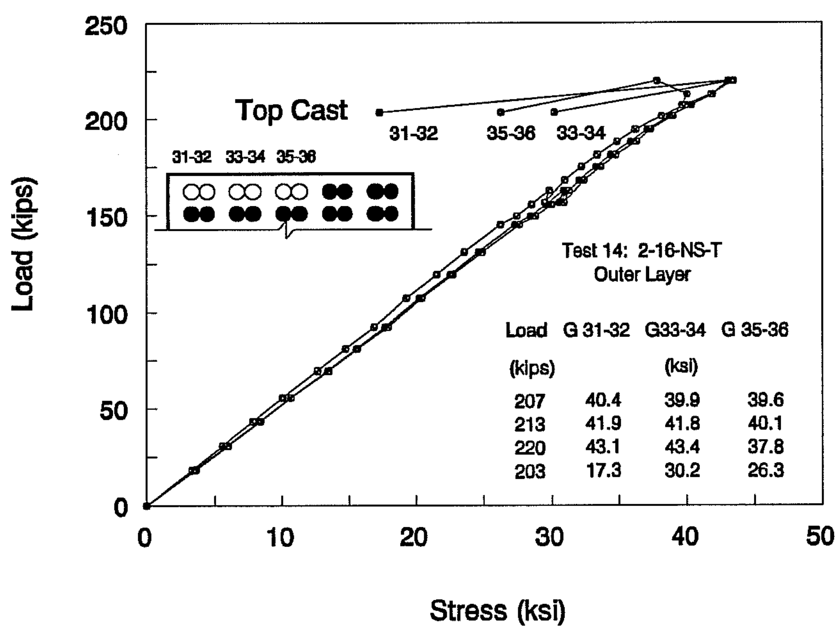
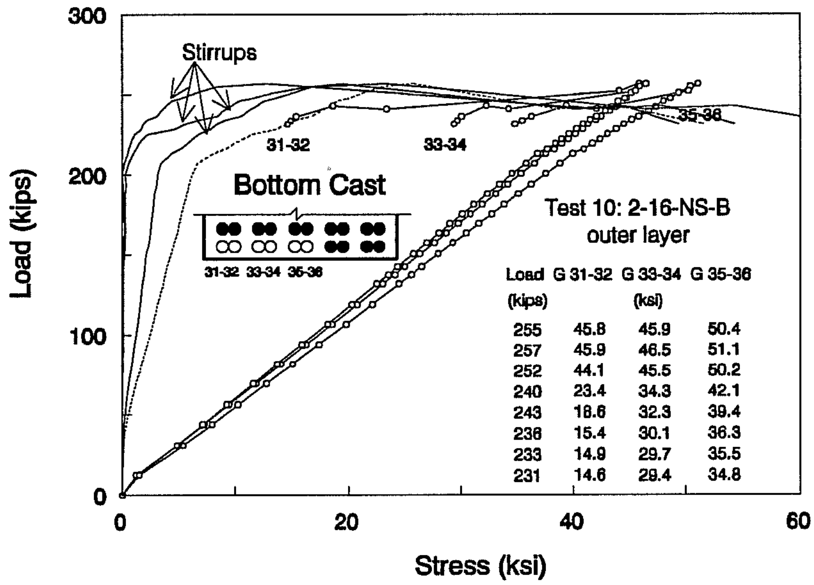


Figure 4.13 **Load versus stress of bundled bars**
(Test 14: 1-16-NS-T)



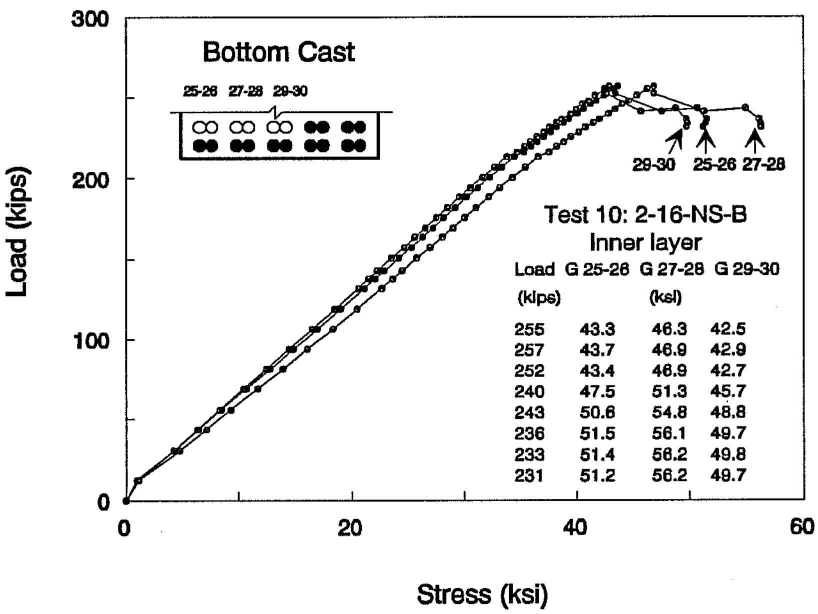


Figure 4.14 **Load versus stress of bundled bars (Test 10: 2-16-NS-B)**

4.3 Tests with Shear

Four tests were carried out with shear acting on the test region. For these four tests, load was moved to produce shear in the test region. Among these four tests, three tests failed in a manner other than bond failure; only one test showed bond failure in the test region.

4.3.1 Tests without Transverse Reinforcement

There were two tests without transverse reinforcement. In the one with a single layer of top cast bundled bars (1-24-S-T), the distance between the loading point and support (shear span) was 52 inches. The other test had two layers of bottom cast bundled bars (2-24-S-B) with a shear span of 68 inches. The results of these two tests were similar. There was a teflon sheet at either end of the test region to separate the anchorage zone from the unbonded region. The presence of the teflon sheet greatly reduced the shear capacity of the beam at that section. As the load increased, diagonal shear cracks appeared first at the corner of the upper teflon sheet and extended to the corner of lower teflon sheet as shown in Figure 4.15. This caused the end concrete block to be sheared off the beam. Six #6 bars provided flexural capacity in this region in both tests. As the load increased, the diagonal shear crack widened and the 6 #6 bars should have yielded before the test bars failed in anchorage as will be explained further in chapter 5. When the load reached ultimate, the stress in the longitudinal bars was much lower than predicted. An examination of the test region showed that concrete cover split in the one layer test and part of the concrete cover split in the two layer test. The failure mode for one layer of bars is shown in Figure 4.16 and the failure mode for two layers of bars is shown in Figure 4.17.

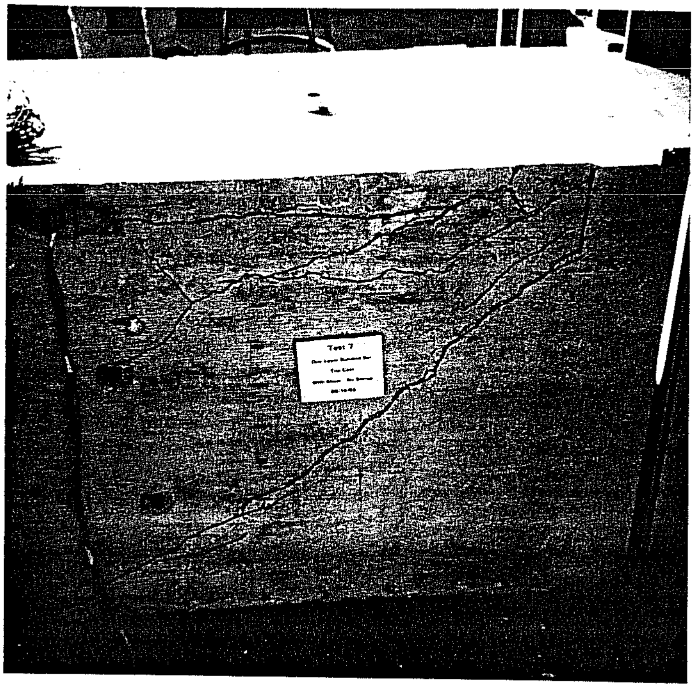


Figure 4.15 Cracking from teflon sheet -- bars without transverse reinforcement

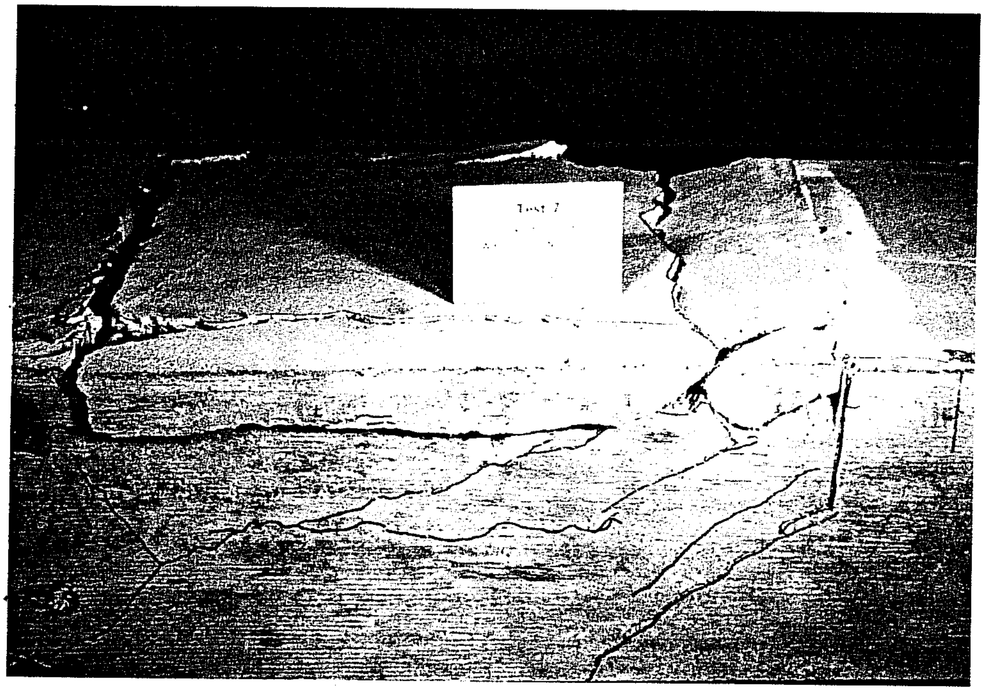


Figure 4.16 Failure mode of one layer of bundled bars without transverse reinforcement, shear in anchorage zone (Test 7 1-24-S-T)

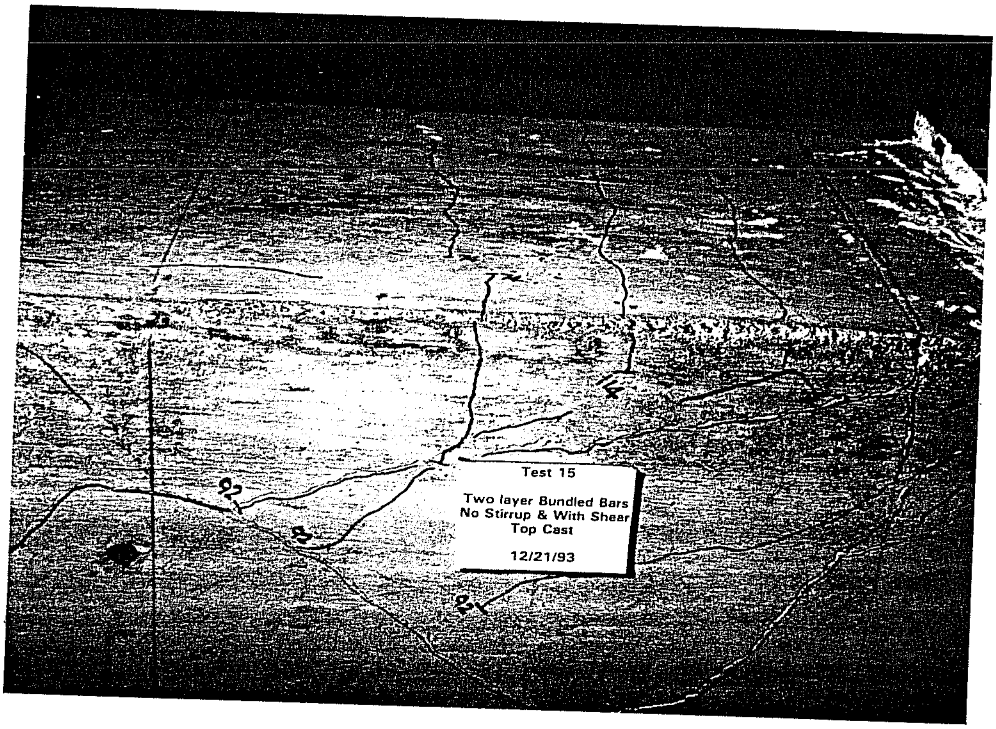


Figure 4.17 Failure mode for two layers of bundled bars without transverse reinforcement-- shear in anchorage zone (Test 15 2-24-S-B)

4.3.2 Test with Transverse Reinforcement

Two tests with transverse reinforcement in the test region were conducted. The first had one layer of top cast bundled bars (1-16-S-T) with a shear span of 52 inches. The second had two layers of bottom cast bundled bars (2-16-S-B) with a shear span of 68 inches. The tests were quite similar to those without transverse reinforcement, except that two diagonal shear cracks appeared in the test region as shown in Figure 4.18. The transverse reinforcement efficiently controlled the width of the shear crack. This was reflected in the test of two layers of bundled bars, in which the stress in the longitudinal bars and stirrups was much higher than that in the test without stirrups, and a bond failure was observed. In the test of one layer of bundled bars, the stress in the longitudinal bars at failure was still much lower than expected. An inspection of the specimen after failure showed that some of 6 # 6 bars did not have sufficient development length to transfer the load. The second shear crack (the one at the left in Figure 4.18) crossed through the development length of the 6 #6 bars and reduced the development length. The section failed when no more load could be transferred due to a bond failure of the additional 6 #6 transfer bars. The failure mode for one layer of bars is shown in Figure 4.19 and the failure mode for two layers of bars is shown in Figure 4.20. Load-stress curves for the test 2-16-S-B are shown in Figure 4.21. No other curves are shown because the results are not considered to represent a bond failure mode.

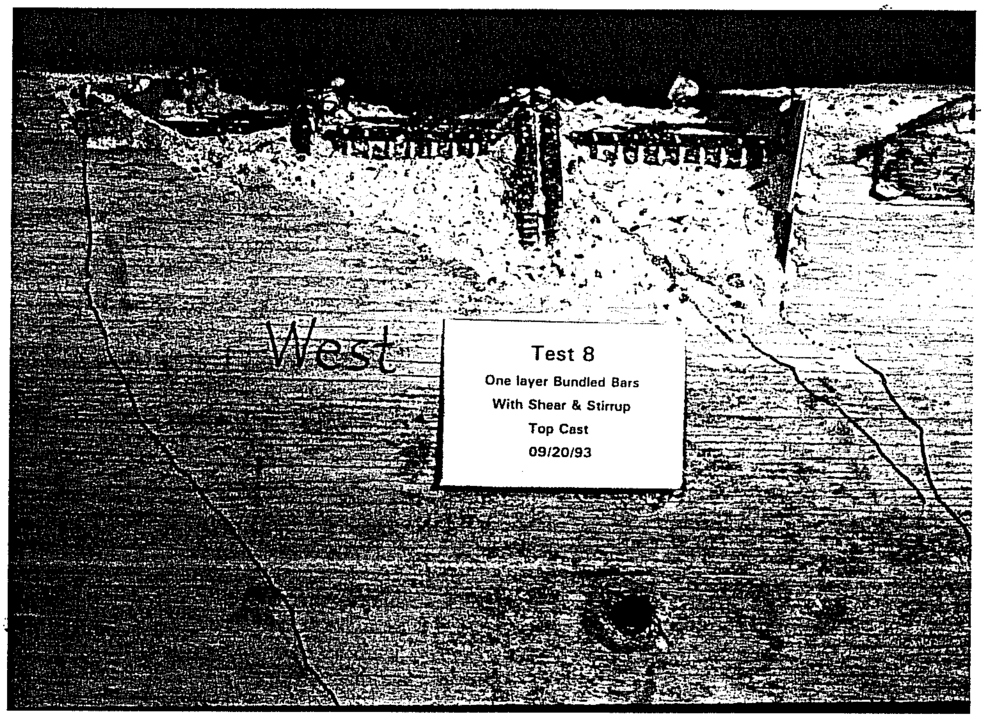


Figure 4.18 Failure at teflon sheet--bars with transverse reinforcement

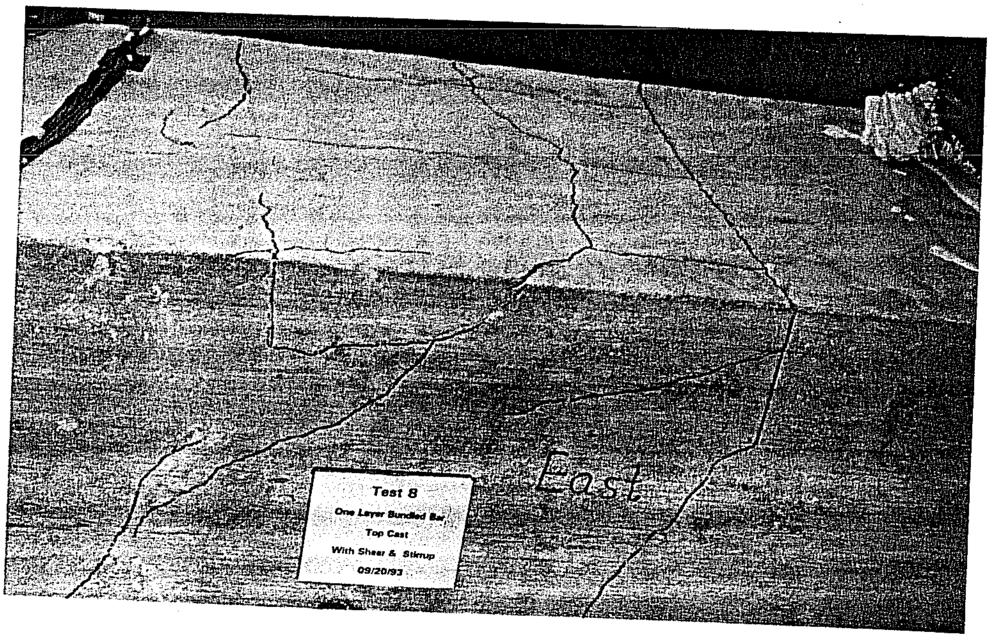
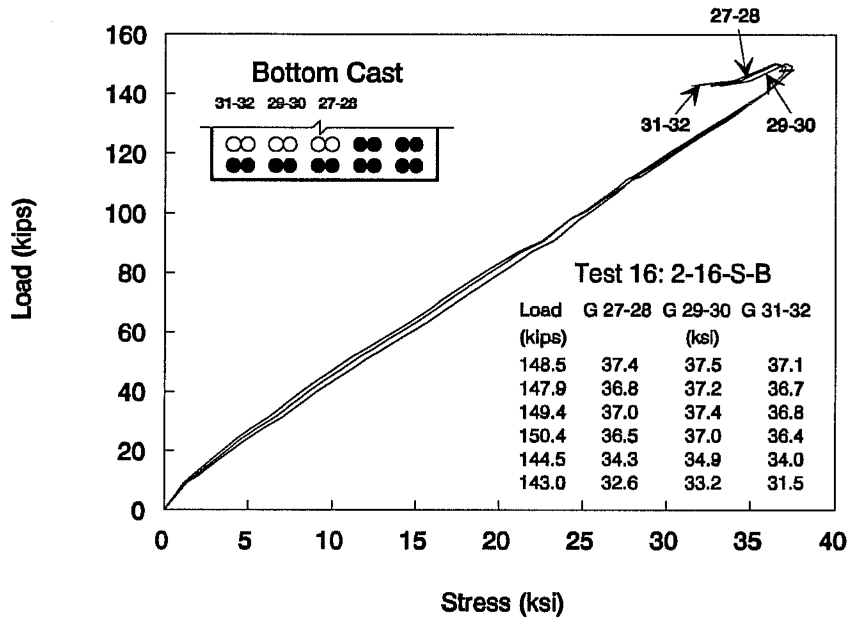
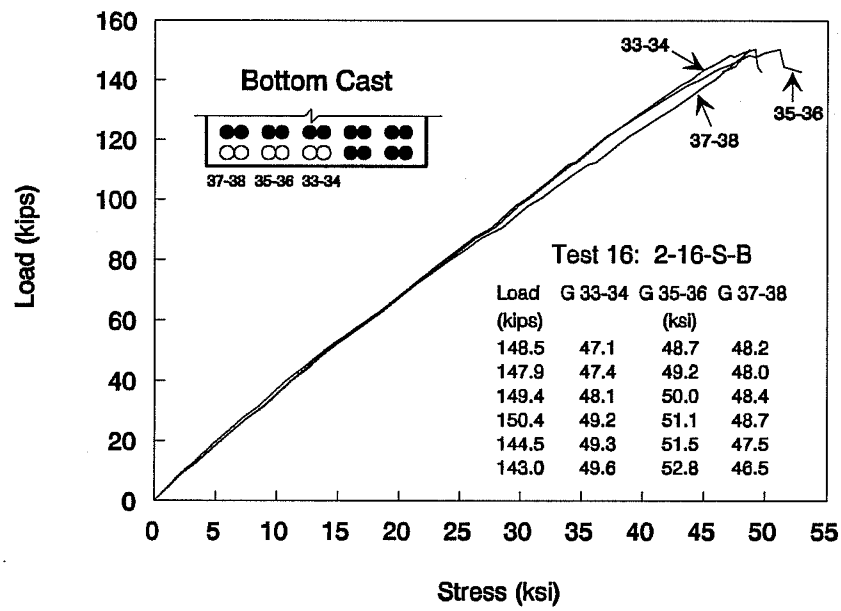


Figure 4.19 **Failure mode of one layer of bundled bars with transverse reinforcement -- shear in anchorage zone (Test 8 1-16-S-T)**

Figure 4.20 **Failure mode of two layers of bundled bars with transverse reinforcement, --Shear in anchorage zone (Test 16 2-16-S-B)**



Load (kips)	G 27-28	G 29-30 (ksi)	G 31-32
148.5	37.4	37.5	37.1
147.9	36.8	37.2	36.7
149.4	37.0	37.4	36.8
150.4	36.5	37.0	36.4
144.5	34.3	34.9	34.0
143.0	32.6	33.2	31.5



Load (kips)	G 33-34	G 35-36 (ksi)	G 37-38
148.5	47.1	48.7	48.2
147.9	47.4	49.2	48.0
149.4	48.1	50.0	48.4
150.4	49.2	51.1	48.7
144.5	49.3	51.5	47.5
143.0	49.6	52.8	46.5

Figure 4.21 Load versus stress of bundled bars --Shear in test region (Test 16: 2-16-S-B)

CHAPTER 5
COMPARISON AND DISCUSSION

5.1 Introduction

Test results are summarized and compared in this chapter. The stress in bundled bars was determined from strain gages on the bars. The load monitored by a pressure transducer and the load calculated using the measured bar strains are compared in Table 5-1. Table 5-1 shows that the calculated loads are very close to the measured load and demonstrates the reliability of the strain gage data. The large difference in the values for Test 12 was carefully examined but it was not possible to find a reason for the high loads determined from strains.

The measured bond strength is taken as the average bond stress at peak load and is compared with the value calculated by Equation 2.9. Both values are shown in Table 5-2. The effects of transverse reinforcement, casting position, layering, and shear are evaluated quantitatively. To compare

Table 5.1 Comparison of measured load and the load calculated from strain gages

Test No.	Specimen number	Calculated Load (kips)	Measured Load (kips)	Errors
5	1-24-NS-T	140.0	142.0	1.4%
12	1-24-NS-B	163.0	135.0	20.7%
9	1-16-NS-T	126.5	131.8	4.0%
13	1-16-NS-B	170.0	168.0	1.2%
11	2-24-NS-T	269.0	255.5	5.3%
6	2-24-NS-B	232.0	218.0	6.4%
14	2-16-NS-T	205.4	219.7	6.1%
10	2-16-NS-B	246.4	256.5	3.9%
15	2-24-S-B	110.8	114.6	3.3%
16	2-24-S-B	145.1	149.4	2.9%

Table 5.2 Summary of Test Results

Test No.	Specimen	Concrete Strength (psi)	Face cover (in)	Measured Bar Stress (ksi)		Index of Trans Rein. K_{tr}	Outer Layer				Inner layer			
				Outer Bar	Inner Bar		Bond Stress U_T (psi)	Test Bond Factor $U_T/\sqrt{f'_c}$	Calcu-lated Bond Factor $U_c/\sqrt{f'_c}$	Ratio of U_T/U_C	Bond Stress U_T (psi)	Test Bond Factor $U_T/\sqrt{f'_c}$	Calsu-lated Bond Factor $U_c/\sqrt{f'_c}$	Ratio of U_T/U_C
Tests without shear														
5	1-24-NS-T	2920	1	42.4			331	6.13	5.20	1.18				
12	1-24-NS-B	4210	$1^1/_8$	51.4			410	6.32	7.30	0.87				
9	1-16-NS-T	2920	1	37.1		3.5	435	8.05	8.56	0.94				
13	1-16-NS-B	2500	$1^1/_8$	54.1		3.5	660	13.20	11.54	1.14				
6	2-24-NS-B	2920	1	47	37.9		367	6.80	6.76	1.01	296	5.48	6.76	0.81
11	2-24-NS-T	4200	$1^1/_2$	52.2	42.7		408	6.29	6.74	0.93	334	5.15	6.74	0.76
10	2-16-NS-B	2920	1	47	43.7	3.5	551	10.20	11.04	0.92	512	9.48	11.04	0.86
14	2-16-NS-T	2550	1	40.8	32.7	3.5	478	9.47	8.68	1.09	383	7.59	8.68	0.87
									Average = 1.01 Standard Deviation = 0.11				Average = 0.83 Standard Deviation = 0.04	
Tests with shear														
7	1-24-S-T*	2640	1	30.6			239	4.65	5.20	0.89				
8	1-16-S-T*	2650	1	22.3		3.5	261	5.08	8.56	0.59				
15	2-24-S-B*	2730	1	34.6	25.8		270	5.17	6.76	0.77	202	3.86	6.76	0.57
16 [1]	2-16-S-B*	2730	1	48	36.5	3.5	563	10.77	11.04	0.97	428	8.19	11.04	0.74

[1] Only specien which failed in bond. No average values or standard deviation given

the influence of these variables, in some tests, the measured stresses of two-bar bundles were adjusted by a factor based on Equation 2.9 to account for differences in the thickness of cover, anchorage length, and concrete strength. The bond failure mechanism in the two layer case is discussed and the problems encountered in the tests with shear are explained.

5.2 Bond Strength of Two-Bar Bundles

In this study, the bond strength of multiple two-bar bundles in one and two layers was examined. Non-bundled bar tests with a comparable cross section area to the two-bar bundle in one and two layers will be carried out in a companion study. In this section, the measured bond strength of two-bar bundles is compared with Equation 2.9 which provides an estimate of the bond strength of non-bundled bars.

Test data and calculated values are summarized in Table 5.2. In this table the measured bond strength is compared with the value computed using Equation 2.9 for individual #6 bars. The measured bond strength was based on the stress in the bars at peak load. Data from eight tests without shear is in one group and that from four tests involving shear is in another group.

In the tests without shear, the bond strength of one layer of bundled bars and the bond strength of the outer layer of bundled bars in the two layer case are comparable and are tabulated together. The average ratio of measured bond strength to calculated bond strength is 1.01 with a standard deviation of 0.11. Equation 2.9 provided an accurate estimate of the bond strength of the single and two layers of two-bar bundles tested in this program.

The bar surface area in contact with concrete is not changed in the two-bar bundles. As a result, there is no significant difference in the bond mechanism between two-bar bundles and non-bundled bars. The test results showed that the two bars in a bundle worked together and there was no relative displacement between the bars. The stresses of the bars within a bundle were close enough to assume that the bars worked as a unit. The results indicated that a two-bar bundle is an efficient way of bundling.

The average ratio of measured bond stress at peak load to calculated bond stress is 0.83 with a standard deviation of 0.04 for the inner layer of bundled bars in the two layer case. But this does not necessarily mean that the bond strength of the inner layer of bundled bars is less than that of the outer layer of bars because bond failures did not occur at the same time in the inner and outer layers as will be explained in detail in section 5.6.2.

For tests involving shear, only data from Test #16 resulted in a measured bond strength that was close to the calculated value. Data from the other three tests showed large inconsistencies between measured and calculated values. Further analysis of the specimen design and performance showed that the failure modes of these three tests were other than bond failure in the test region as will be explained in section 5.5.

5.3 Effect of Transverse Reinforcement

In Equation 2.3, the bond strength is determined by summing the contribution of concrete cover and transverse reinforcement. The contribution of transverse reinforcement is a function of the area of transverse reinforcement "A_{tr}", the spacing of the transverse steel "s", and the bar diameter of the anchored bars. The calculation of "A_{tr}", for typical cases, is shown in Figure 5-1 (a). The factor "u_{tr}" is expressed by the following:

$$u_{tr} = \frac{A_{tr} f_y}{500\ s\ d_b} \tag{5.1}$$

The force which can be developed by the ties is $A_{tr} f_y$. Although it is assumed that the ties are at yield for simplification in design, stresses in the stirrups are usually much lower than yield even at failure. Stirrups usually begin to pick up stress near the point of bond failure as the concrete strain in the test region reaches the tension fracture strain. The stress in the stirrups at the point of bond failure is limited by the tension fracture strain of the concrete. For example, if the strain at the concrete tension fracture is about 0.0002, the corresponding stress in the stirrups would only be about 5.8 ksi. To increase the contribution of transverse reinforcement to bond strength, the strain of the transverse reinforcement at bond failure would have to be increased. For two layers of bundled bars, there are two potential splitting planes crossed by transverse reinforcement. Therefore, the strain of the transverse reinforcement should be at least doubled at failure. Actually, the transverse reinforcement should be more efficient in confining the inner layer of bars as verified by the measured stress on the stirrups in the tests of one layer and two layers of bars (shown in Figure 5.2). While the

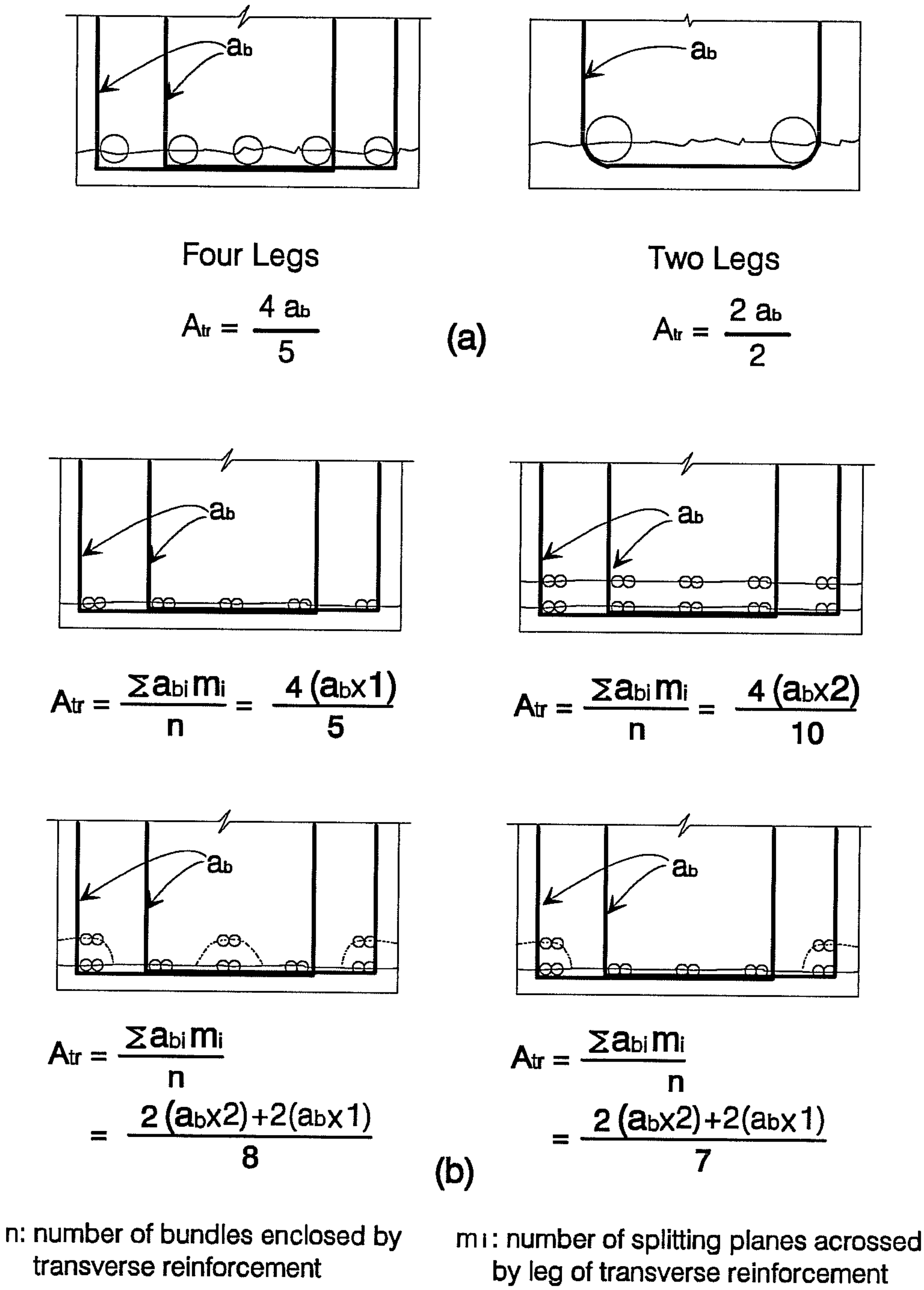
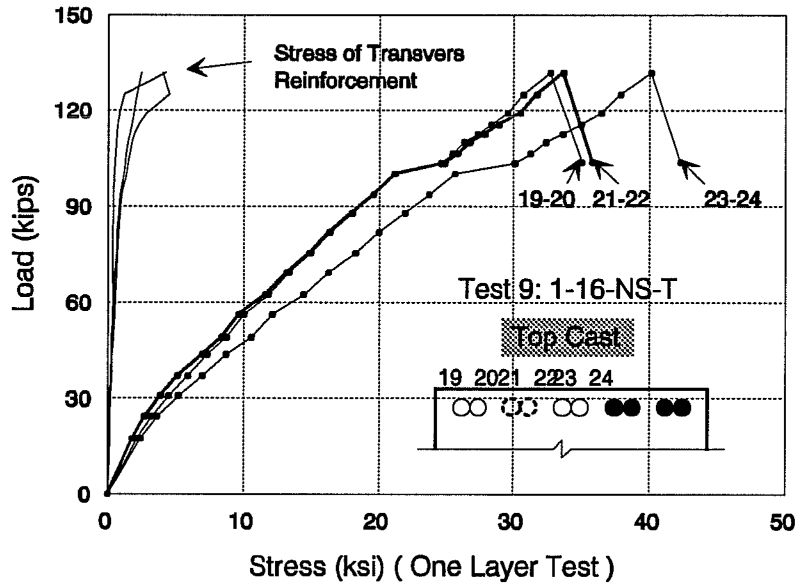


Figure 5.1 **Definition of the Area of Transverse Reinforcement " Atr"**



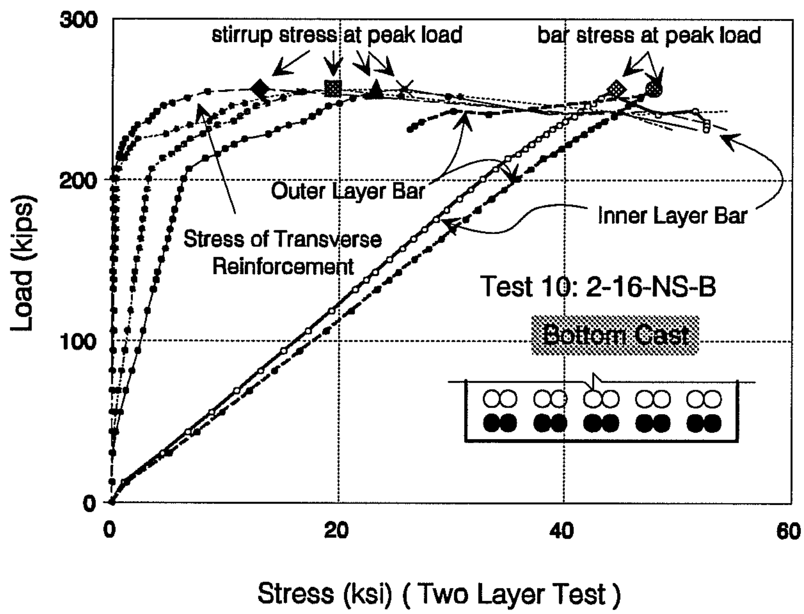


Figure 5.2 **Comparison of Stirrup stress for one and two layers of bundled bars**

stress in the stirrups for one layer of bars was about 5 ksi at the peak load, the stress in the stirrups for two layers of bars was about 20 ksi. In the test with two layers of bars, splitting in the planes of both inner and outer layers was noted at failure.

Equation 2.9 is a regression formula based on test data. To include the confinement of transverse reinforcement on two layers of bundled bars, Equation 5.1 needs to be modified. The easiest way may be to change the method of calculating A_{tr}. For the data in this test program, the value of A_{tr} were defined as shown in Figure 5.1 (b) and used in Table 5.2.

(1) For one layer of bundled bars: A_{tr} is defined as the total area of transverse reinforcement divided by the number of bundles which are enclosed by ties rather than the number of bars.

(2) For two layers of bundled bars: A_{tr} is defined as the summation of the product of each leg area times the number of splitting planes crossed by transverse reinforcement, divided by the number of bundled bars which are enclosed by transverse reinforcement.

The calculated values in Table 5-2 based on these definitions, compare satisfactorily with measured values for the tests without shear.

5.4 Influence of Casting Position

Previous research showed that poorer concrete below top cast bars will be detrimental to bond strength. The low quality of the concrete below the top cast bars is a result of segregation of aggregates, accumulation of air voids, and bleed water below the top bars as shown in Figure 5.3. In

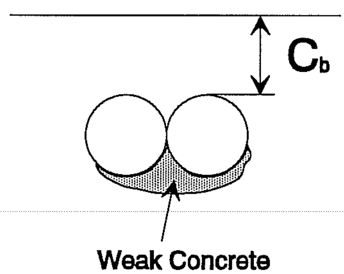


Figure 5.3 Inferior concrete below top casting bars

this program the beams were 30 in. deep. Both the outer and inner layer of the upper bars in the beam had more than 12 in. of fresh concrete placed below the bars and can be regarded as top cast reinforcement according to AASHTO specifications. Each test geometry was cast in the top and bottom positions to provide a basis for evaluation. Table 5-3 compares the difference in bond strength of top and bottom cast bars.

Some values were modified to account for the difference in concrete cover and embedded length. For example: the average measured stress of bundled bars is 54.1 ksi for Test 13. But the concrete cover for this test is $1^1/_8$ inches. The stress of 54.1 ksi needs to be modified to correspond with the 1 in. cover. Using Equation 2.6, the bond factor is 11.04 for 1 in. cover and 11.54 for $1^1/_8$ in. cover in Test 13. The measured stress is modified by the ratio of 11.04 to 11.54:

$$(f_s)_{mod} = 54.1 \times \frac{11.04}{11.54} = 51.7\ ksi$$

The average ratio of bond strengths for bottom cast versus that for top cast bars is 1.23 (Table 5.3). The standard deviation for the casting position factor is 0.23, which is quite large but typical of variations reported by other investigators. In the AASHTO Code the factor for top cast bars is 1.4 and is probably appropriate considering the small number of two-bar bundles tested in this program and the large standard deviation found.

5.5 Influence of Shear

In this program, the tests involving shear were not very successful due to a problem in the design of specimen. Only one of the four tests resulted in a bond failure in the test region.

5.5.1 Tests without Transverse Reinforcement

In the shear test, there were two tests without transverse reinforcement; one with one layer of top cast bundled bars (1-24-S-T) and the other with two layers of bottom cast bundled bars (2-24-S-B). As shown in Table 5.2, the measured bond strength in both tests was much lower than that calculated by Equation 2.9.

Table 5.3 Comparison of Casting Position

Test No.	Specimen Number	Casting Position	Anchor Length L_d (in)	Face Cover C_c (in)	Concrete Strength f'_c (psi)	Stress of Bar[1] Measured (ksi)	Stress of Bar[1] Modified[2] (ksi)	Bond Factor[3]	$\frac{U_{Bot}}{U_{Top}}$
5	1-24-NS-T	Top	24	1	2920	42.2		6.13	0.93
12	1-24-NS-B	Bottom	23.5	$1^1/_8$	4210	51.4	47.5	5.72	
9	1-16-NS-T	Top	16	1	2920	37.1		8.05	1.51
13	1-16-NS-B	Bottom	16	$1^1/_8$	2500	54.1	51.7	12.12	
11	2-24-NS-T	Top	24	$1^1/_2$	4,200	52.2	40.2	4.85	1.40
6	2-24-NS-B	Bottom	24	1	2920	47.0		6.80	
14	2-16-NS-T	Top	16	1	2,550	40.8		9.47	1.08
10	2-16-NS-B	Bottom	16	1	2920	47.0		10.2	
					Standard Deviation = 0.23			Avg.	1.23

Note: [1] The stress of one layer of bars or the stress of the outer layer of bars in two layer case
[2] Bar stress normalized for cover thickness (1"), anchorage length (24")
[3] Bond factor = $u_T / \sqrt{f'_c}$

A review of each crack pattern and performance of the specimen indicated that the teflon sheet at either end of the test region may have triggered an early failure. The depth of the teflon sheet was 4 inches. The teflon sheet significantly reduced the shear capacity of the beam at that section. When the load was applied, a diagonal shear crack appeared first at the corner of the upper teflon sheet and extended to the lower teflon sheet. This caused the end portion of the beam to be sheared off as shown in Figure 4.15. The applied load then had to be transferred to the beam by a group of 6 #6 bars which were added for flexure strength. Analyzing a free body diagram shown in Figure 5.4, it was found that the stress in 6 #6 bars was close to yield at the peak load. The failure of these two tests was caused by early development of a diagonal shear crack and subsequent yielding of the 6 #6 bars rather than by bond failure of the anchored bars.

5.5.2 Tests with Transverse Reinforcement

Two specimens reinforced with transverse ties involved shear. One was Test 8 with one layer of top cast bundled bars (1-16-S-T) and the other was Test 16 with two layers of bottom cast bundled bars (2-16-S-B). As explained above, the existence of the teflon sheet reduced the shear capacity of the beam at those sections. The diagonal shear crack appeared very early in the test, and the end block tended to be sheared from the beam. In both tests, two groups of transverse reinforcement crossed the shear crack. The transverse reinforcement controlled the shear crack and enabled the 6 #6 bars to transfer load to the beam as shown in Figure 5.5. In Test 16, the stress in the longitudinal bundled bars and transverse ties was much higher than those in the test without transverse reinforcement, and a bond failure was produced in the test region.

Unfortunately, in Test 8, some of the 6 #6 bars were shorter than others and they did not have enough development length when the unexpected cracks appeared early on the beam. As shown in Figure 4.18, a second shear crack (the left one) appeared in the test region. This crack extended to the anchorage zone of the 6 #6 bars and might have caused some reduction in anchorage length. The measured stress in the longitudinal bundled bars was very low at failure. In retrospect, the teflon sheet at the load end of the anchored bars could have been eliminated without detriment to the test and might have enabled the bars to reach bond strength before a shear failure stopped the test.

In summary, only one of the four tests involving shear truly reflected the bond strength. In Test 16, the test with shear, the bond factor for the outer layer of bars was 10.77, and the bond factor

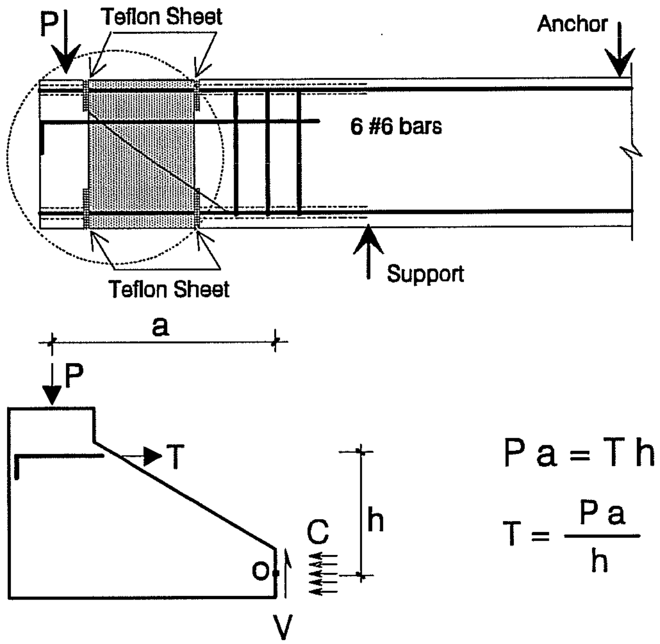


Figure 5-4: **The Separating Diagram of End Concrete Block under the Test without Transverse Reinforcement**

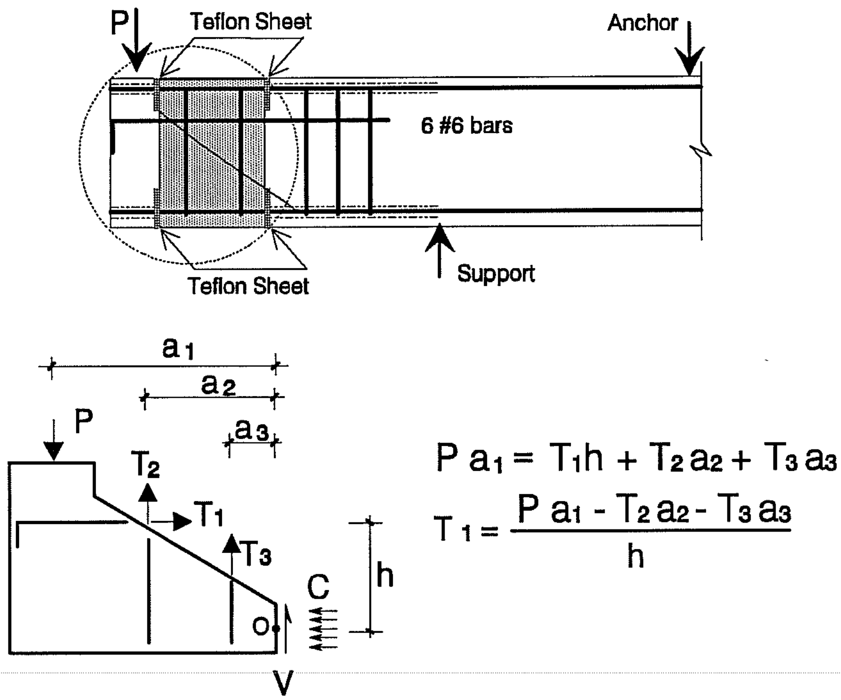


Figure 5-5: **The Separating Diagram of End Concrete Block under the Test with Transverse Reinforcement**

was 10.2 in the companion test (test #10), without shear (Table 5-2). Shear acting on the test region should not be considered to be beneficial to the bond strength of bundled bars even though data showed that the bond strength with shear was a little higher than that without shear. Although there was only one test involving shear, it was significant that shear did not seem to affect the bond strength. It should also be noted that the test value of $10.77\sqrt{f_c}$ was almost equal to the calculated bond stress of $11.04\sqrt{f_c}$. The results of tests with shear demonstrated once again the importance of careful detailing in regions of high shear and bond and of the beneficial aspects of transverse reinforcement.

5.6 One Layer versus Two Layers of Bundled Bars

One of the principle reasons for conducting this program was to investigate the performance of multiple layers of bundles. The bond strength of the outer layer bars in the two layer case is very close to that of the case with one layer of bars. The stress ratio of the inner layer of bars to the outer layer of bars was only 0.8 at the peak load when it should have be 0.91 based on strain compatibility by assuming a linear strain gradient in the section. The bond failure mechanics for two layers of bundles can be explained from the observed test results.

5.6.1 Bond Strength

The bond strength of bundled bars in one layer and the bond strength of the outer layer of bars in the two layer case are summarized in Table 5.4. The bond strength in these two cases can be compared using the bond factor $u/\sqrt{f_c}$. In this table, some of the measured bar stresses are modified as explained in section 5.4. The average bond factor ratio of single layers to the outer layers in two layer cases is 1.04. This indicates that the bond strength of outer layer of bars in two layer case is very close to the bond strength of bars in one layer. But the standard deviation of 0.18 is fairly large. This is mainly due to the large difference in cover thickness and concrete strength between the tests and the small number of tests conducted.

Table 5.4 Comparison of One and Two Layers of Bar

Test No.	Specimen Number	Casting Position	Anchor. Length L_d (in)	Face Cover C_c (in)	Concrete Strength f'_c (psi)	Stress in Bar [1] Test (ksi)	Stress in Bar [1] Mod.[2] (ksi)	Stress in Bar [1] Bond Factor $U_T/\sqrt{f'_c}$	$\frac{U_{T\ one\ layer}}{U_{T\ outer\ layer}}$
5	1-24-NS-T	Top	24	1	2920	42.2		6.13	1.26
11	2-24-NS-T	Top	24	$1^1/_2$	4200	52.2	40.2	4.85	
12	1-24-NS-B	Bottom	$23^1/_2$	$1^1/_8$	4210	51.4	47.5	5.72	0.87
6	2-24-NS-B	Bottom	24	1	2920	47.0		6.80	
9	1-16-NS-T	Top	16	1	2920	37.1		8.05	0.85
14	2-16-NS-T	Top	16	1	2550	40.8		9.47	
13	1-16-NS-B	Bottom	16	$1^1/_8$	2500	54.1	51.7	12.12	1.19
10	2-16-NS-B	Bottom	16	1	2920	47.0		10.2	
					Standard Deviation = 0.18			Avg.	1.04

Note: [1] The stress of one layer of bars or the stress of the outer layer of bars in two layer case
[2] Bar stress normalized for cover thickness (1"), anchorage length (24")

5.6.2 The Mechanism of Bond Failure in Two Layers of Bundled Bars

In the two layer case, the stress in the outer layer of bars was always greater than the stress in the inner layer of bars from the beginning of the test to the peak load. There was little effect of the inner layer of bars on performance of the outer layer of bars. Furthermore, the bond strength of the outer layer of bars in the two layer case was close to the bond strength of one layer of bars. It may be concluded that there is no deterioration in bond strength for the outer layer of bundled bars. The failure mechanism of an outer layer of bundled bars is the same as that of one layer of bundled bars which is caused by the radial pressure on the concrete due to the tension force in the bundled bars.

To analyze the failure mechanism of the inner layer of bars, it may be helpful to review the failure mechanism in Specimen #1, in which there was a piece of plywood sheet backing the test region as explained in chapter 3. The reason that the bond strength in two layer case was much lower in Specimen #1 was that the tension force from the outer layer of bars had to be transferred across the plane of the inner layer of bars. The combination of shear and radial pressure from the inner layer caused an early failure in the plane of inner bars. In Specimen #2, the tension force in the outer layer of bars was carried by bearing force on the teflon sheet as shown in Figure 5-6. There was much less, if any, shear stress transferred across the plane of the inner bars. The splitting failure in the inner plane was due to the radial pressure caused by the inner layer of bars alone.

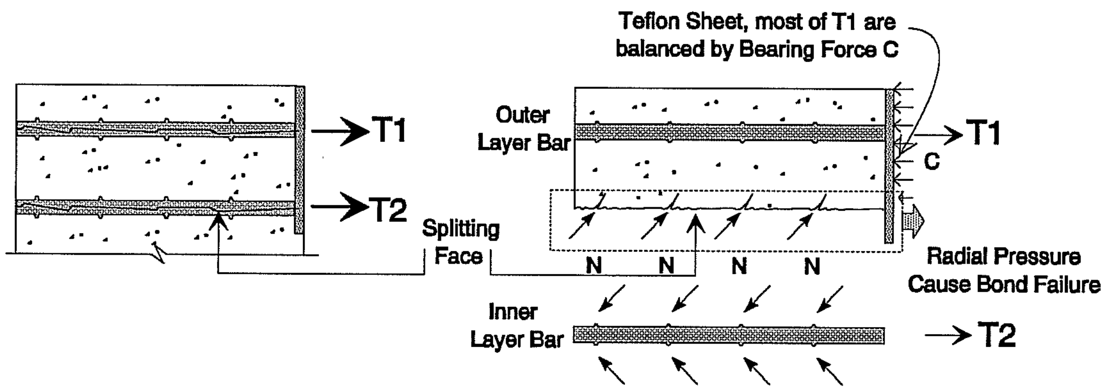


Figure 5-6: The Bond Failure Mechanism of Two Layer of Bundled Bars (with teflon Sheet)

The failure process may be explained more clearly by the load-stress relationship. The stress in the outer layer of bars was about 20 percent higher than that in the inner layer. However, a horizontal crack always appeared first at the end of the beam in the plane of the inner layer of bars. After that, a second horizontal crack appeared in the plane of the outer layer. At first, it was hard to explain this phenomenon because the stress in the outer layer of bars was higher than that in the inner layer of bars. After carefully studying the stress history of the inner and outer layer of bars from the peak load to failure, it appeared that the crack in the plane of the inner layer of bars occurred after the peak load was reached. The load-stress curves (Figures 5.7 through 5.10) for four tests showed that before the peak load was reached, the stress in the outer layer of bars was always greater than the stress in the inner layer; after the peak load, the stress in the outer layer of bars began to decrease and the stress in the inner layer of bars began to increase.

At peak load, splitting cracks in the plane of the bars were not visible. Usually, there were three longitudinal cracks on the cover above the middle and two corner bundles. These three cracks weakened the confinement of the outer layer of bars and relieved the stress in the outer layer of bars. To maintain the load at same level, stresses in the outer layer of bars were transferred to the inner layer of bars. Before bond failure, the stresses in the inner layer of bars usually were close or exceeded the stresses in the outer layer of bars. As the force transfer changed, a horizontal crack probably appeared in the plane of the inner layer of bundled bars.

The load changed after the peak load as stress was being transferred from the outer layer of bars to the inner layer of bars. The moment arm of the outer layer of bars was larger than that of the inner layer of bars. To maintain load, the stress on the inner bars would have to increase, and the results showed such an increase. The second crack that formed in the plane of outer bars was likely due to the slip of bars rather than the initial splitting that resulted from radial tension. This phenomenon was more obvious in the tests with transverse reinforcement (test 10, test 14) than in the tests without transverse reinforcement (test 6, test 11). In the tests without transverse reinforcement, cracking among the inner and outer planes of bars and subsequent failure occurred almost simultaneously. For that reason, stress transfer could not be monitored by the strain gages.

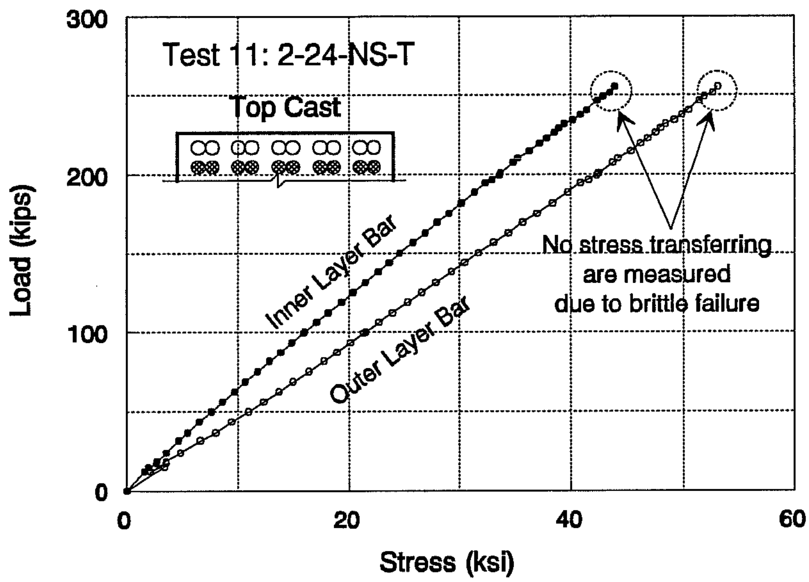


Figure 5.7 **Average stress of outer and inner layer Bars (Two layers of bars without transverse reinforcement, Top Cast)**

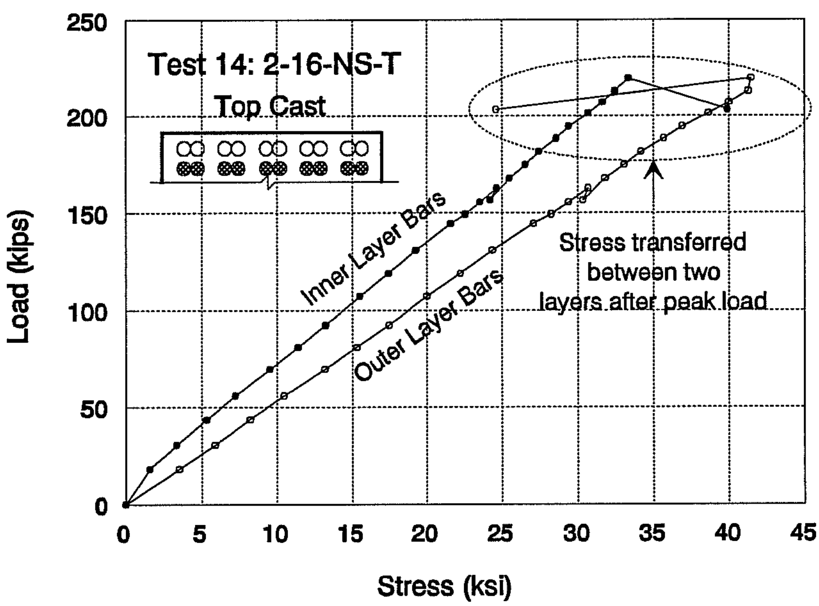


Figure 5.8 **Average stress of outer and inner layer bars (Two layers of bars with transverse reinforcement, Top Cast)**

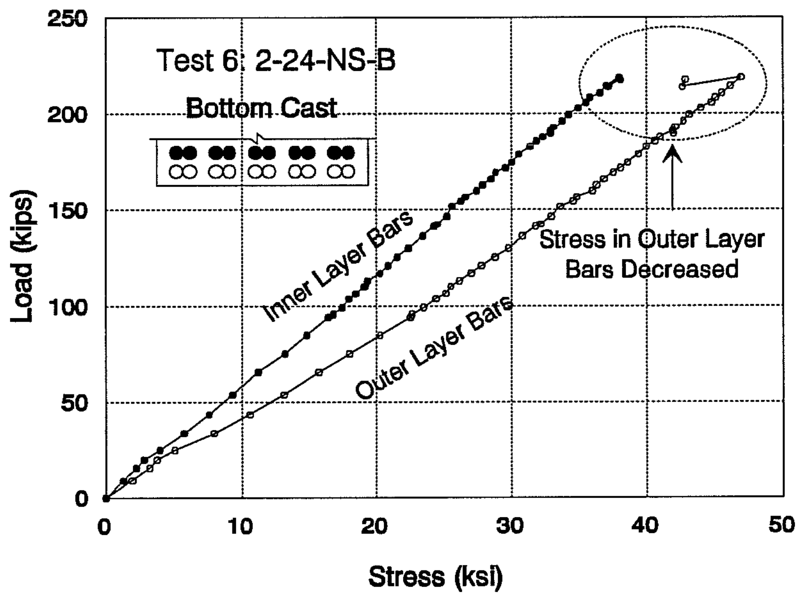


Figure 5.9 Average stress of outer and inner layer of bars (Two layer of bars without transverse reinforcement, bottom cast)

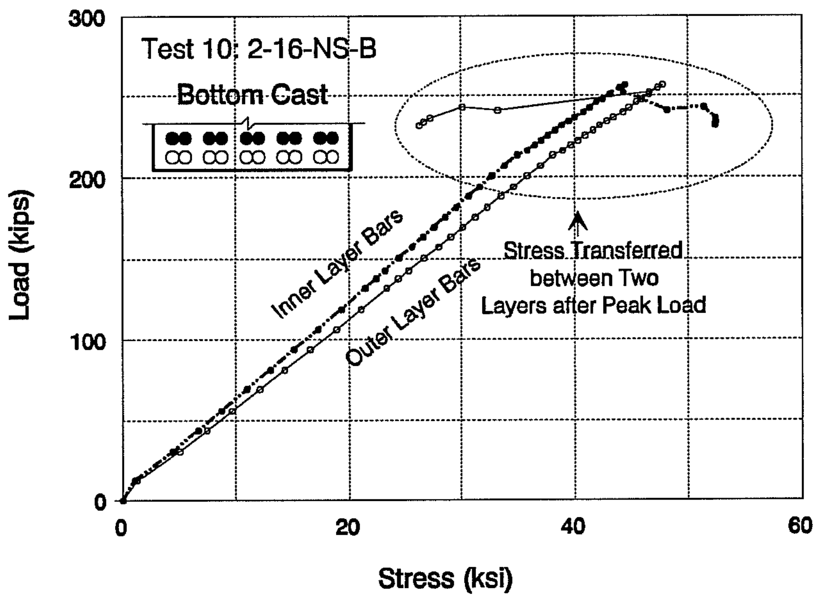


Figure 5.10 Average stress of outer and inner layer of bars (Two layer of bars with transverse reinforcement, bottom cast)

The stress of bundled bars at first cracking may also help to explain some observations from the tests. Usually, the first longitudinal crack appeared just before the bond failure however there were some differences in the stress level at which the first longitudinal crack appeared for the different bar configurations. Table 5.5 summarizes the stress level in the bars at first cracking. The data in Table 5.5 shows two trends in the cracking stress level:

(1) The load, at which the first longitudinal crack appeared, was close (3 of the 4 pairs of tests) to the peak load in the tests with transverse reinforcement than in the test without transverse reinforcement. This demonstrated the effect of confinement from the transverse reinforcement.

(2) The load, at which the longitudinal crack appeared, was close to the peak load in the tests of one layer of bars than in two layers. This supports the idea that there will be stress transfer between the outer layer of bars and the inner layer of bars, because the early appearance of longitudinal cracks will result in stress transfer from the outer to the inner layer of bars.

Table 5.5 Stress at First Longitudinal Cracking

Test No.	Specimen Number	Cracking Stress (ksi)	Peak Loading Stress (ksi)	Stress Ratio at Cracking Load to Peak Loading
5	1-24NS-T	38.9	42.2	0.92
12	1-24-NS-B	44.9	51.4	0.87
9	1-16-NS-T	NA(1)	37.1	≈ 1.0
13	1-16-NS-B	NA(1)	54.1	≈ 1.0
11	2-24-NS-T	42.5	52.2	0.81
6	2-24-NS-B	33.6	47.0	0.71
14	2-16-NS-T	27.1	40.8	0.66(2)
10	2-16-NS-B	41.0	47.0	0.87

(1) Crack appeared at failure

(2) This test was repeated because a shear failure occurred in the beam. The shear span was increased to reduce shear in the second test.

To summarize, when the maximum stress in the outer layer of bars was reached (at peak load), the outer layer of bars were near bond failure. However, the stress in the inner layer of bars was close to (test 14) or exceeded (test 10) the maximum stress in the outer layer of bars at the end of the test. The progression of failure was such, that the inner plane failed first and promoted a failure in the outer plane. This leads to the following two conclusions:

(1) Tension in the outer layer of bars had little, if any, effect on the bond strength of the inner layer of bars. The failure of the inner layer of bars is caused by radial pressure due to the tension force in the inner layer of bars alone.

(2) The bond strength of the inner layer of bars may even be a little higher than that of outer layer of bars due to the additional confinement provided by the outer layer of bars.

The maximum stress in the outer and inner layer of bars does not occur at same time. Therefore, slightly higher bond strength in the inner layer of bars does not lead to an increase in strength at the section. In practice, design must be based on the peak load. At peak load, the stress in the inner layer of bars is less than that of the outer layer. Therefore, the stress levels at the peak load are relevant for design and are discussed in greater detail below.

5.6.3 Stress Level in the Outer Layer and Inner Layer of Bars at Peak Load

Table 5.6 shows a comparison of the bar stress in the outer and inner layers at peak load. Based on the four tests conducted, the average measured stress ratio of the inner to the outer layer bars is 0.84, while the calculated stress ratio based on strain compatibility within the section is 0.91. Figure 5.11 shows the relationship of these stress ratios versus load. Except for Test 10, there is nearly a 10% difference between the two ratios. In Figure 5.11, the calculated stress ratio is shown only at peak load because several assumption must be made to determine the stress ratio. One possible approach is to assume that plane sections remain plane, but the unbonded portion of the bars in the test beam may nullify this assumption. In the unbonded region, the stress in the bars is constant but the moment varies. Since the slip of the bars at the loaded end was not measured, it is hard to determine if the difference is mainly due to the configuration of the test specimen or if there are other reasons for the difference.

Table 5.6 Comparison between Measured and Calculated Stress Ratios of Outer layer to Inner Layer of Bars at Peak Load

Test No	Specimen Number	Outer Layer Stress (ksi)	Inner Layer Stress (ksi)	Measured ratio of Inner layer to Outer layer	Calculated ratio of inner layer to outer Layer
6	2-24-NS-B	47.0	37.9	0.81	0.90
11	2-24-NS-T	52.2	42.7	0.82	0.91
10	2-16-NS-B	47.0	43.7	0.93	0.90
14	2-16-NS-T	40.8	32.7	0.80	0.91
			Average:	0.84	0.91

5.7 Stress Distribution across Section

In all tests, there were five two-bar bundles per layer. Stresses of the bundled bars varied across the section as shown in Figures 5.12 and 5.13. The stress distribution is plotted for several different load levels. The X-axis denotes the location of strain gages relative to the edge of the beam. The stress is the average of the measured stresses within a bundle. The distribution across the section was assumed to be symmetric since only three bundles were gaged.

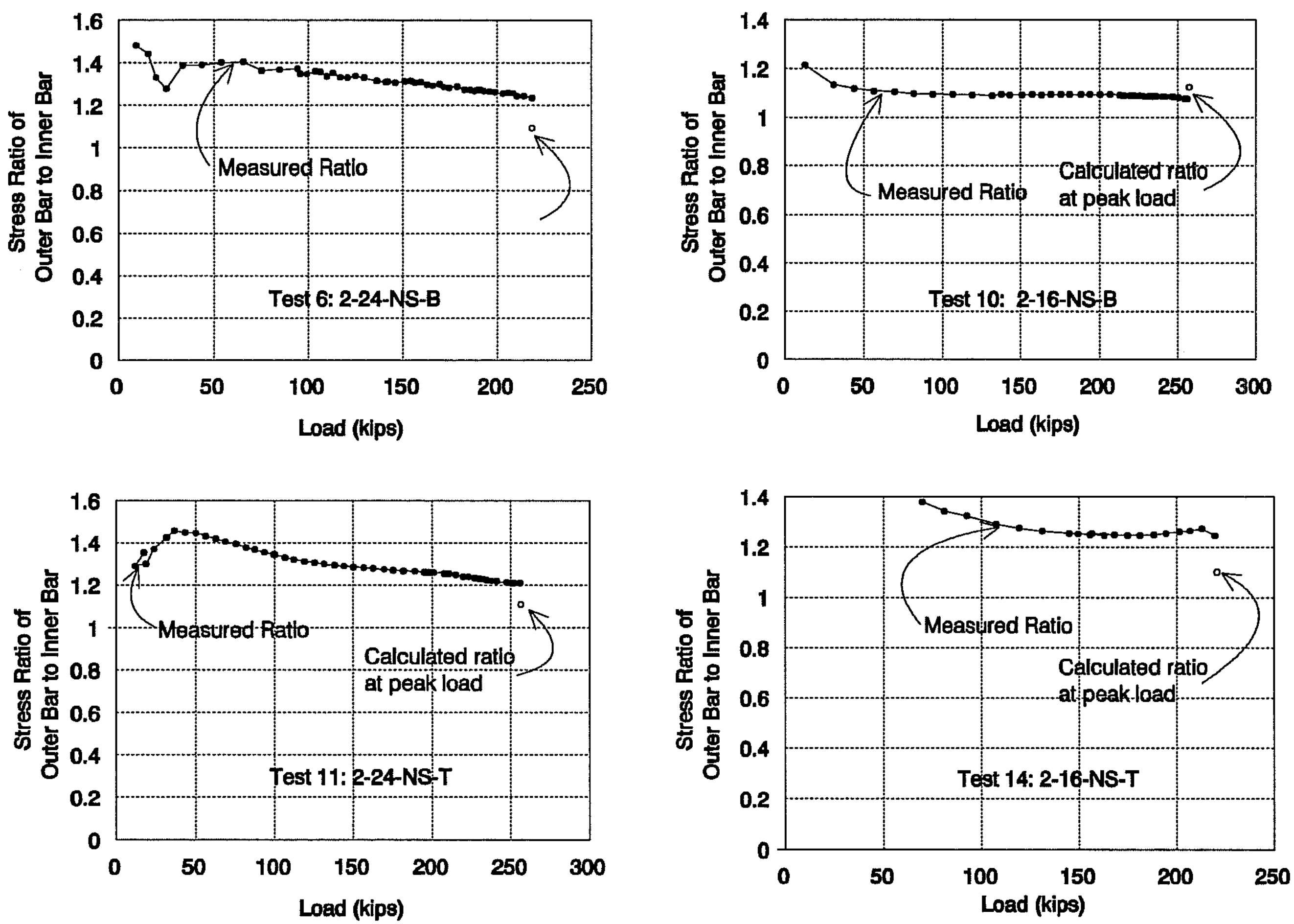
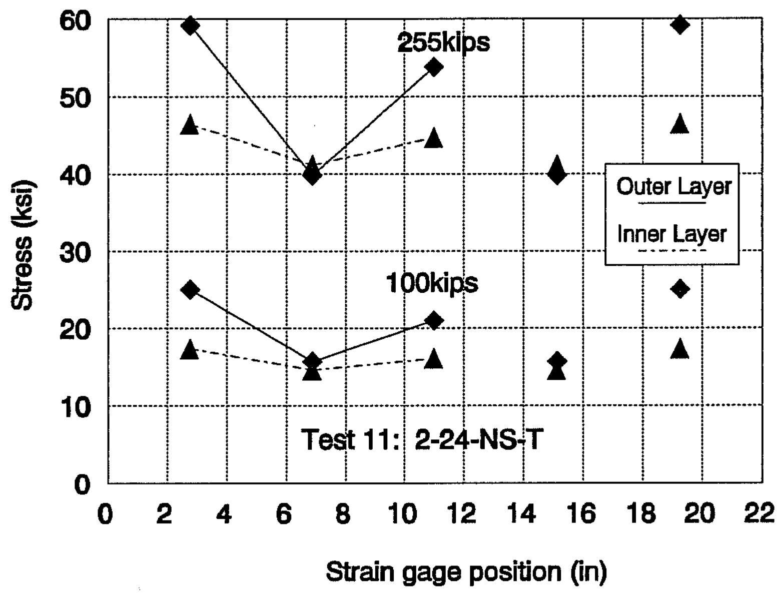


Figure 5.11 **Stress ratios between outer layer and inner layer of bars**



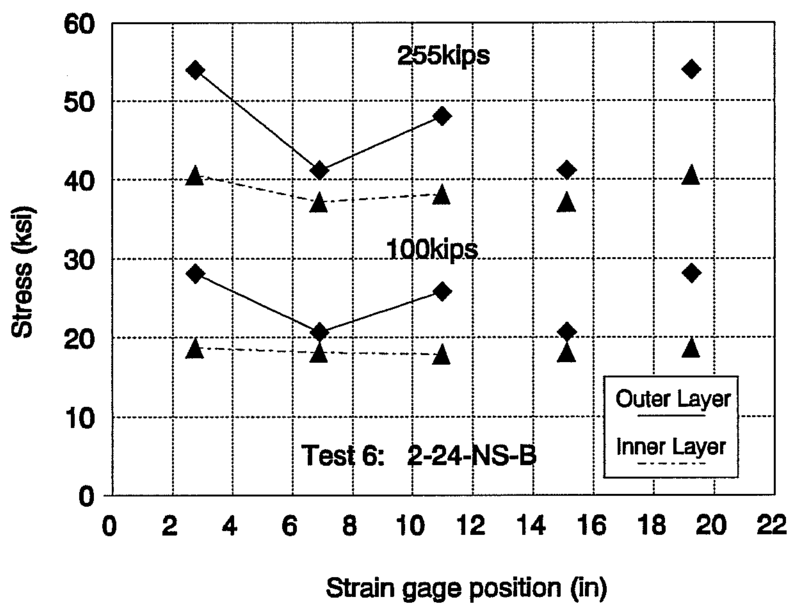
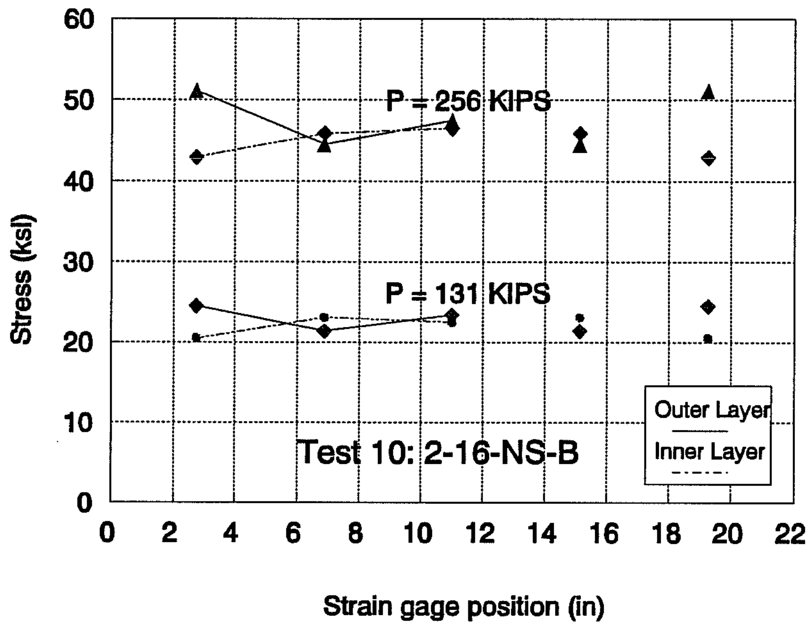


Figure 5.12 **Measured stress distribution across section**
(Tests without transverse reinforcement)



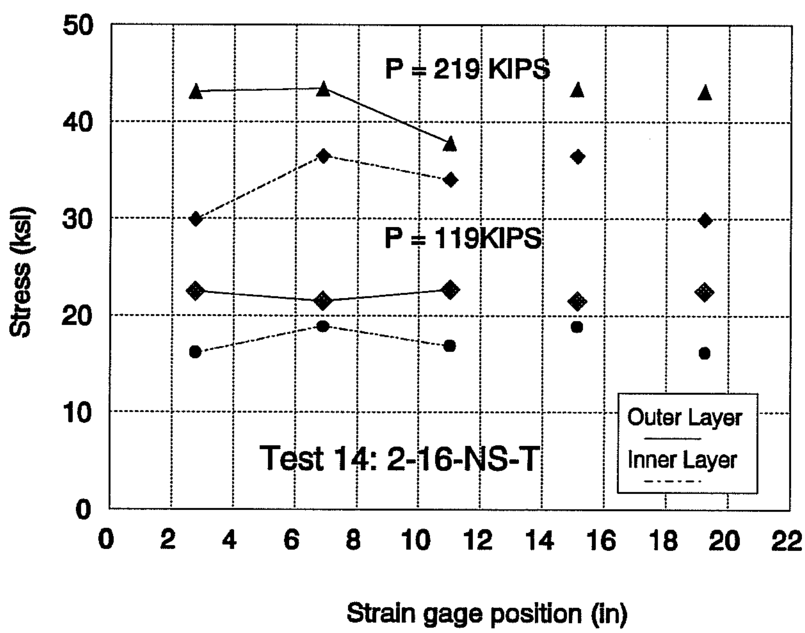


Figure 5.13 Measured stress distribution aross section
(Tests with transverse reinforcement)

There was no consistent pattern of stress distribution across the section. In some tests the stress in the corner bundles was larger than the stress in the middle bundle while in others, the reverse was true. Generally, the stresses were more uniform in the inner layer than in the outer layer. The confinement of the inner layer bars was relatively uniform, and the layer was less affected by construction errors such as changes in cover, or spacing between the longitudinal bars and the transverse ties.

The stresses were also more uniform at lower loads than at higher levels. Usually, if the stress in one bundle was lower than other bundles initially, it remained lower throughout the test. Failure in these tests was brittle and there was no opportunity for stresses to be redistributed among the individual bundles especially when there was no transverse reinforcement.

5.8 Development Length for Two-bar Bundles

Neither AASHTO nor ACI318-89 mention the two-bar bundles in calculating the development length. Measured bundled bar stress and the stress calculated based on current AASHTO and ACI Codes are compared in Table 5.7 and 5.8. The measured stresses in some tests in these two tables are normalized to 1 inch cover thickness based on Equation 2.9 as explained before. The cover factor for development length is based on diameter of single #6 bar. The calculated stress is based on the following equation:

$$f_s = \frac{l_d \sqrt{f_c'}}{0.04\, A_b\, F_c\, F_p}$$

where l_d = anchorage length. (24", 16")

A_b = cross section area of single bar

F_c = cover modification factor

F_p = casting position factor

Table 5.7 and 5.8 show that both AASHTO and ACI codes are unconservative for bars without stirrups and appropriate for bars with stirrups. Therefore, section 12.4.2 of ACI 318-89 Code needs to be modified as follow: "for determining the appropriate modification factors, a unit of <u>two or more</u> bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area." (New words underlined)

Measured and calculated bar stresses based on proposed method are compared in Table 5.9. The proposed method provides a conservative method of calculating the development length for two-bar bundles.

Table 5.7 Comparison of stress between measured and calculated (AASHTO)

Test No.	Conc. Stren. f_c' (psi)	Meas. Stress (ksi)[(1)]	Calculated Stress			Ratio
			Cover factor F_c	Casting posi. F_p	Stress (ksi)	$\frac{(f_s)mea.}{(f_s)cal.}$
12 (1-24-NS-B)	4210	47.5	1.0	1.0	88.5	0.54
6 (2-24-NS-B)	2920	47.0	1.0	1.0	73.7	0.64
5 (1-24-NS-T)	2920	42.4	1.0	1.4	52.6	0.81
11 (2-24-NS-T)	4200	40.2	1.0	1.4	63.1	0.64
Average (without transverse reinforcement)						0.66
13 (1-16-NS-B)	2500	51.4	1.0	1.0	45.5	1.13
10 (2-16-NS-B)	2920	47.0	1.0	1.0	49.1	0.96
9 (1-16-NS-T)	2920	37.1	1.0	1.4	35.1	1.06
14 (2-16-NS-T)	2550	40.8	1.0	1.4	32.8	1.24
Average (with transverse reinforcement)						1.10

(1) measured stresses are normalized for 1" cover thickness

Table 5.8 Comparison of stress between measured and calculated (ACI)

Test No.	Calculated Stress (ACI)			Ratio (f_s)mea. (f_s)cal.
	Cover factor F_c	Casting Posi. F_p	Stress (ksi)	
12 (1-24-NS-B)	1.4	1.0	63.2	0.75
6 (2-24-NS-B)	1.4	1.0	52.6	0.89
5 (1-24-NS-T)	1.4	1.3	40.5	1.05
11 (2-24-NS-T)	1.4	1.3	48.5	0.83
Average (without transverse reinforcement)				0.88
13 (1-16-NS-B)	1.4	1.0	32.5	1.58
10 (2-16-NS-B)	1.4	1.0	35.1	1.34
9 (1-16-NS-T)	1.4	1.3	27	1.37
14 (2-16-NS-T)	1.4	1.3	25.2	1.62
Average (with transverse reinforcement)				1.48

Table 5.9 Comparison of stress between measured and calculated (proposed method)

Test No.	Calculated Stress (Proposed method)			Ratio (f_s)mea. (f_s)cal.
	Cover factor F_c	Casting Posi. F_p	Stress (ksi)	
12 (1-24-NS-B)	2.0	1.0	44.3	1.07
6 (2-24-NS-B)	2.0	1.0	36.9	1.27
5 (1-24-NS-T)	2.0	1.3	28.3	1.50
11 (2-24-NS-T)	2.0	1.3	34.0	1.18
Average (without transverse reinforcement)				1.26
13 (1-16-NS-B)	1.4	1.0	32.5	1.58
10 (2-16-NS-B)	1.4	1.0	35.1	1.34
9 (1-16-NS-T)	1.4	1.3	27	1.37
14 (2-16-NS-T)	1.4	1.3	25.2	1.62
Average (with transverse reinforcement)				1.48

(1) Diameter based on equivalent two-bar area =1.06"

CHAPTER 6
SUMMARY AND CONCLUSION

6.1 Summary

Test Program The primary objective of this test program was to analyze the bond strength of multiple two-bar bundles considering the effect of the following variables: number of layers of bars, casting position, amount of transverse reinforcement, and level of shear in anchorage zone. The configuration of reinforcement was based on typical TxDOT pier cap detail. Sixteen tests were conducted. Four tests were conducted to develop the test procedure. Of the 12 tests evaluated in this project, eight had no shear acting on the test regions, and four were subjected to shear and anchorage stresses. Only one of the four tests involving shear failed in bond in the test region. The other three failed by a combination of shear and load transfer problems. Therefore, the conclusions are based on eight tests without shear and one with shear.

Mode of Failure As planned, the measured stress in all the bundled bars was below the yield stress, and all the bond failures were face-and-side split modes. The longitudinal cracks always appeared first above the two corner bundles. In the tests with transverse reinforcement, a third longitudinal crack appeared above the middle bundle, since this bundle was not confined by a stirrup leg. In the tests of two layers of bundled bars, a horizontal crack appeared first in the plane of the inner layer of bars at the free end of the anchored bars. If the load was maintained, a second horizontal crack formed in the plane of the outer layer of bars. As soon as the second horizontal crack formed, the specimen failed.

In the two layer case, the stress in the outer layer of bundled bars was higher than that in the inner layer. Since bond failure was brittle, there was no stress redistribution between the two layers untill the peak load was reached. Near peak load, the outer layer of bars was close to bond failure. If the load was maintained at that level, part of the stress in the outer layer of bars was transferred to the inner layer as the confinement for the outer bars was lost due to splitting. As the stress of inner layer of bars increased, bond failure in the inner layer was produced. Because the moment arm of the inner layer of bars was smaller than that of the outer layer, load began to decrease when the stress was transferred from the outer layer to the inner layer.

Most of the forces in the outer layer of bundled bars were balanced by the bearing force in the outer layer of concrete, so there was little shear stress, if any, transferred to the plane of inner layer of bundled bars. Bond failure occurred in both inner and outer planes. Failure mechanisms in both planes were due to concrete splitting produced by tension in bars.

6.2 Conclusions

1. Bond Strength of Two-Bar Bundle

Geometrically, bundling two bars did not significantly change the bond surface area between concrete and reinforcement. The bars within a bundle always worked together. The bond failure mechanism was same as that of an individual bar. The formula developed by Orangun et. al., to estimate the bond strength of an individual bar, was found to be suitable for estimating the bond strength of two-bar bundles.

2. Bond Strength of Two-Bar Bundles in Two Layers

Bond failure occurred in both inner and outer planes. There was not much difference in the failure mechanism between one layer of bundled bars and two layers of bundled bars. At peak load the bond strength of the outer layer of bundled bars in the two layer case was very close to the bond strength of one layer of bundled bars. Finally, when the inner layer of bars failed, bond strength of the inner layer was slightly higher than that of the outer layer, probably due to additional confinement provided by the outer layer. However in a real structure, only peak load can be relied on. At that point, the stress of the inner layer of bars, in the tests reported here, was only about eighty percent of the outer layer of bars and the bond strength of the outer layer of bars was about the same as the bond strength of one layer of bars.

3: Casting Position

The top casting position had a detrimental effect on bond strength for the two-bar bundles. The average bond strength ratio of bottom cast bars to top cast bars was 1.23 based on the eight tests without shear. However, because of the high standard deviation (23%), the 1.4 top casting position factor seems appropriate.

4. Effect of Transverse Reinforcement

There were two potential splitting planes crossed by the transverse reinforcement in the tests with two layers of bundled bars. Stirrups were more effective in restraining splitting in the two layer case than in the one layer case. The equation for individual bar proposed by Orangun et. al. was effective for calculating the contribution of transverse reinforcement to the bond strength of two-bar bundles. The equivalent area of stirrups "A_{tr}" is redefined by the following: "A_{tr}" is equal to the summation of the product of each leg area times the bar layers crossed by the leg, divided by the number of bundles enclosed by the ties.

5. Effect of Shear

Although four tests were constructed, difficulties with the test procedure led to premature shear failures before a bond strength of the anchorage zone was reached. The one test which was performed as desired revealed no measurable effect of shear on the bond strength of bundled bars.

6. Design Consideration

In calculating the development length for two-bar bundles, the basic development length is the same as single bars, but the modification factor for cover and spacing should be based on a single bar of a diameter derived from the equivalent two-bar area.

6.3 Other Issues

The tests did not provide a clear indication of reason for the difference in the observed stress ratio of the inner and outer layers at the peak load. The difference may have been due to the lack of compatibility due to the unbonded bar length or to a more complex force transfer mechanism when multiple layers and various bar patterns are used. Further experiments are needed to define the performance. However, the test results indicated that beam specimen with partially unbonded bars was efficient in studying bond strength.

BIBLIOGRAPHY

1. C.O. Orangun, J.O. Jirsa, J.E. Breen, "The Strength of Anchor Bars: A Reevaluation of Test Data on Development Length and Splices", Research Report 154-3F, Center for Highway Research, The University of Texas at Austin, 1975
2. J. O. Jirsa, J. E. Breen, "Influence of Casting Position and Shear on Development and Splice Length -- Design Recommendation", research Report 242-3F, Center for Transportation Research, The University of Texas at Austin.
3. P. M. Ferguson, "Bond Stress -- the State of the Art, ACI Committee 408", ACI Journal, October, 1960, PP. 1161-1188
4. Mark A. Thompson, J. O. Jirsa, ect, "The Behavior of Multiple Lap Splice in Wide Sections", Research Report 154-1, Center for Highway Research, The University of Texas at Austin,January, 1975
5. Andrew John Zekany, "The Influence of Shear on the Strength of Lapped Splices", Unpublished Master's Thesis, The University of Texas at Austin, May 1980
6. Bent Steen Andreasen, "Anchorage Tests with Ribbed Reinforcing Bars in More than One Layer at a Beam Support", Department of Structural Engineering, Technical University of Denmark, Series R, No.239
7. N. W. Hanson, Hans Reiffenstuhl, "Concrete Beams and Columns with Bundled Reinforcement", Journal of the Structural Division, October, 1958
8. Leroy A. Lutz, "Crack Control Factor for Bundled Bars and for Bars of Different Sizes", ACI Journal, January 1974
9. Edward G. Nawy, "Crack Control in Beams Reinforced with Bundled Bars Using ACI 318-71", ACI Journal, October, 1972
10. ACI Committee 318, "Building Code Requirements for Reinforced Concrete and Commentary," ACI Standard 318-89, American Concrete Institute, Detroit, Michigan, 1989.
11. American Association of State Highway and Transportation Offcials, "Standard Specifications for Highway Bridges" 1989

VITA

Weixiong Chen was born in Shanghai, China, on February 23, 1965, the son of Ronqiang Chen and Miaofen Xie. After he graduate in Beijiao High school, Shanghai, in 1982, he entered The Tongji University, Shanghai and majored in structural engineering. As soon as he got a Bachelor of Civil Engineering degree in 1986, he was hired by the Research Institute of Structural Engineering, Tongji University, Shanghai and worked as a teacher and research assistant. In August, 1991 he came to The United States to continue his study. First he was accepted in the Graduated School of The Michigan Technological University, in Honghton, Michigan. In January 1992, he transferred to the Graduate School of The University of Texas at Austin.

Permanent address: 501, 15 Hao, 165 Long
Da lian Road, West
Shanghai, China 200092

This thesis was typed by the author.