BOND STRESS BETWEEN STEEL AND CONCRETE IN TWO-WAY REINFORCED CONCRETE SLABS

Thesis

Submitted to

The School of Engineering of the

UNIVERSITY OF DAYTON

In Partial Fulfillment of the Requirements for

The Degree

Master of Science in Civil Engineering

by

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UNIVERSITY OF DAYTON

Dayton, Ohio

December, 1984

Thesis

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APPROVED BY:

ABSTRACT

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IN TWO-WAY REINFORCED CONCRETE SLABS

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University of Dayton, 1984

Major Professor: Elmer H. Payne

A brief experimental study comparing the pullout force required to cause bond failure in one and two-way reinforcement was attempted. Three different embedment lengths (12", 14", 16") were investigated.

One-way specimens used a single rebar embedded in a 6" diameter by 24" long concrete cylinder and one rebar embedded in 6"x24"x24" concrete blocks. The two-way specimens used two rebars embedded perpendicular to each other in a 6"x24"x24" concrete block. The one-way specimens were tested using a standard universal testing machine. The two-way specimens were tested using a system specially designed for this study.

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ACKNOWLEDGEMENTS

I wish to dedicate this thesis to my Dear Father, Sabeh, and my lovely mother, Affifi, without whose support this dream would have never come true.

I especially would like to acknowledge my advisor, Professor Elmer H. Payne, whose guidance, help and suggestions made this work possible.

I also wish to acknowledge my brother, Johnny; my sisters, Rima, Jeanette, and Aline; and all others who have helped in fulfilling my objectives.

Last, but not least, I would like to dedicate this thesis to a very dear person whom I admire very much, Miss Halla H. Razak.

CHAPTER I

INTRODUCTION

Background

The three most common materials for most civil engineering structures that are built are timber, steel, and reinforced concrete (including prestressed concrete). Reinforcing steel and concrete are used together, thus the design criteria of reinforced concrete differ from those involving only one material.

Reinforced concrete is a union of two materials: plain concrete, which possesses high compressive strength but a low tensile strength, and reinforcing steel rods commonly embedded in the tension zone in the concrete to provide the needed strength in tension. Steel possesses high tensile strength.

Steel and concrete work in combination for several reasons. One of those reasons is bond or interaction between steel and the concrete surrounding it, which prevent slip of the bars relative to the concrete. Also, they have sufficiently similar rates of thermal expansion, that is, 0.0000055 to 0.0000075 for concrete and 0.0000065 for steel per degree fahrenheit (°F) which indroduce negligible stresses between steel and concrete under atmospheric changes of temperatures.

Problem Description

Many civil engineering structures are built of reinforced concrete: bridges, retaining walls, tunnels, tanks, conduits, columns, slabs, and others.

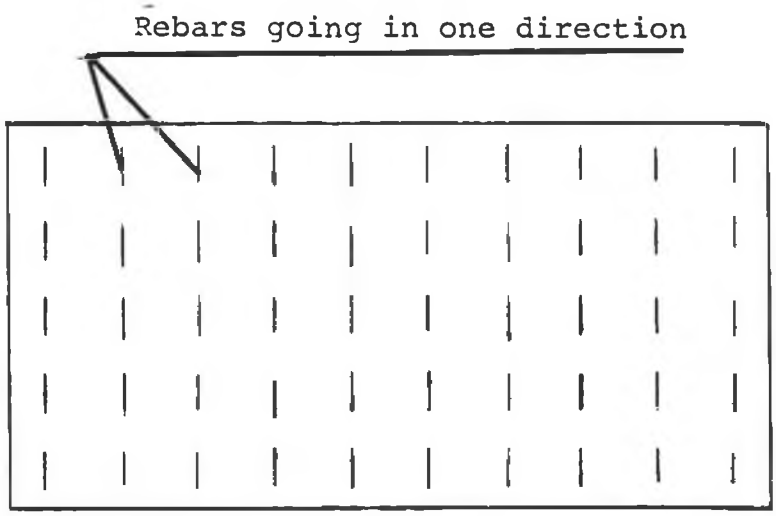
Slabs are classified as one-way and two-way reinforced slabs, depending whether they are reinforced in one or two directions.[1] Slabs are usually reinforced in two directions only when the slab dimensions are square or nearly square (see Figures 1a and 1b).

One- and two-way reinforced slabs have been widely used for the last few decades. As civil engineers, it is always beneficial to get more and more familiar with the behavior of those two methods of reinforcing slabs.

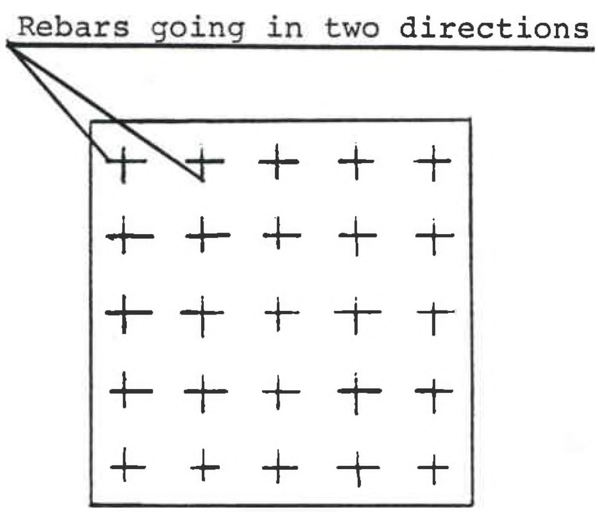
Understanding of reinforced concrete slab behavior is still far from complete; experiments, building codes, and specifications that give design procedures are continually changing to reflect latest knowledge.

In design cases relative sizes of members are needed in the preliminary analysis that must precede the final design, so final conciliation between analysis and design is largely a matter of trial, judgment, and experience.

In this research, an attempt at studying the effect of bond stresses in one- and two-way slabs is conducted.



(a) One-Way Reinforced Slab
Plan View



(b) Two-way Reinforced Slab
Plan View

Figure 1. One- and Two-Way Slab Representation.

Objective

The main objective of this work is to study the bond behavior of two-way reinforcing as compared to the one-way reinforcing in concrete slabs.

The study of this effect includes the bond behavior, pullout resistance, crack formation inside the concrete, and the different modes of failure that the one- and two-way reinforced slabs might be subjected to. Pullout tests were conducted on one- and two-way specimens attempting to attain this goal.

Review of the Literature

The French were the first to make practical use of reinforced concrete in 1867 recognizing many of its potential uses.[2] Nevertheless, ancient Grecian structures have been found which show that builders knew something about reinforced structures.

In 1855, Lambot in France registered the first patent which was on a reinforced concrete beam and a column reinforced with four round iron bars. In England, Wilkinson took out the first patent for a reinforced concrete floor.

In the first decade of the twentieth century, progress in reinforced concrete was rapid. During the 1950's, emphasis was given to studying the behavior of various types of slab floor systems. One-way reinforced slab systems were experimentally studied, particularly with regard to strength, bond, and cracking.

A literature search on bond stress in one- and two-way slabs was done through the University of Dayton's library. The engineering index and a computer search (compendex) were the available sources. The investigation revealed that the bond between deformed reinforcing bars and concrete in one-way slabs has been investigated for a long time and many experimental studies have been conducted regarding one-way reinforcing.

However, no information was found with regard to the bond behavior of two-way reinforcing. This fact suggested that some initial research should be started on this subject.

One important pullout test regarding one-way reinforcing was done in 1958 by Ferguson.[3] This test explained the nature of bond forces between deformed bars and concrete, how they are generated, how they act, and how they cause failure. Also, Ferguson explained the bond stress distribution along reinforcing bars upon application of pullout load and occurrence of bar slip (see Figures 2 and 9). More explanation about Ferguson's work is included in Chapter II.

In 1963, the ACI building code for ultimate strength design proposed a formula to predict the magnitude of the ultimate bond stress. This formula was given as follows:

$$u = \frac{8.0\sqrt{f_c'}}{d} \qquad (1)$$

where:

u is the ultimate bond stress,

f_c' is the concrete compressive strength, and

d is the diameter of the steel bar.

In 1979 Skorobogatov[4] performed a series of tests studying the influence of the geometry of deformed steel on its bond strength in concrete. In these tests, the influence of the magnitude of the slope of the lugs of deformed bars on their bond strength was investigated using different diameter bars. The finding of these tests was that the angles of slope of the lugs did not affect the maximum bond stress, that is the ultimate bond stress.

In the year 1981, an analytical study was done by S. Somayaji and S. P. Shah[5] on bond stress versus slip relationship and cracking response of tension members. They proposed an analytical model to predict the cracking response of concrete members subjected to uniaxial tension. In their analytical model instead of assuming a bond stress versus slip relationship, a function was assumed to represent the bond stress distribution. To check the validity of their analytical model, an experimental investigation was conducted in this regard. Since there was no good agreement between their experiments and the analytical model that they proposed, it was found that their model cannot be considered totally satisfactory.

In 1982, an important test was done by Ralejs Tepfers[6]. This test was very beneficial in explaining the different modes of bond failures and the mechanics of bond formation. He introduced the idea of tensile concrete rings surrounding the reinforcing bar. Tepfers test was accompanied with photographs that illustrated the different types of failure that was encountered in his experiment.

In 1983, the ACI building code for ultimate strength design provides a formula to calculate the required embedment length, ℓ_d.

ACI 12.2.2 states:

$$\ell_d(\text{required}) = \frac{0.04\ A_b f_s}{\sqrt{f_c'}} \tag{2}$$

where:

A_b = the area of steel bar,

f_s = stress in steel bar, and

f_c' = concrete compressive strength.

Summary of Work

Chapter II deals with the mechanics of bond failure. A great deal of explanation and figures, about the nature of bond and their types of failures in one-way slabs are presented. Also, speculations about the load carrying capacity and bond failure in two-way slabs are included.

Testing is the title of Chapter III. Preliminary mix design, one- and two-way tests, are included. Also, discussion of test set-ups, sources of errors, and results are provided in Chapter III.

Chapter IV concludes this work and addresses recommendations for further studies in this field. Also, it presents the limitations and assumptions that were made in this course of study.

CHAPTER II

MECHANICS OF BOND FAILURE

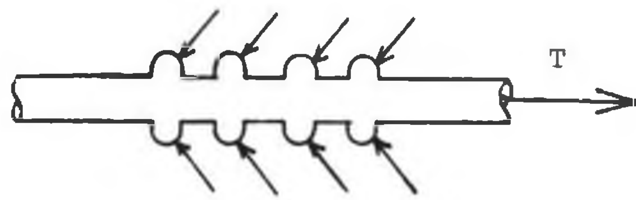
Bond Failure - One-Way

For concrete and steel to work together in a slab it is necessary that stresses be transferred between the two materials. The term "bond" is used to describe the means by which slip between concrete and steel is prevented or at least minimized. Whenever the tensile or compressive stresses in a bar change, bond stresses must act along the surface of the bar to transfer these changes to the concrete. Bond stresses are, in effect, longitudinal shearing stresses acting on the surface between the steel and concrete. They are normally evaluated in terms of pounds per square inch of bar surface and denoted by the symbol u. The bond stress, u, acting as shear between the reinforcing bar and the concrete, gives rise to principal tensile and compressive stresses in the concrete. When the lowest of the shear, principal tensile, or principal compressive strengths is exceeded, changes in the bond conditions occur, which result in failure.

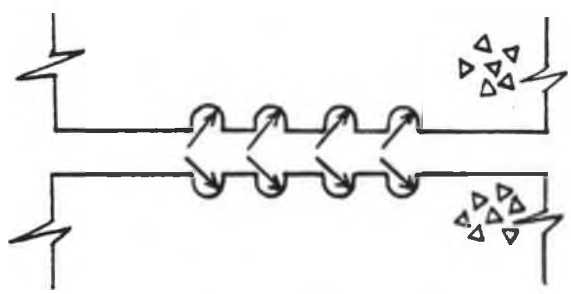
Three types of bond failure can occur.

1. Shear failure along the perimeter of the bar: Usually for very smooth reinforcing bars, the shear strength between the bar and the concrete is the lowest. In this case the shear strength will be the governing factor. Failure in bond along the perimeter of the bar will occur as the bar is pulled out. This type of failure is common among smooth bars with large diameters.

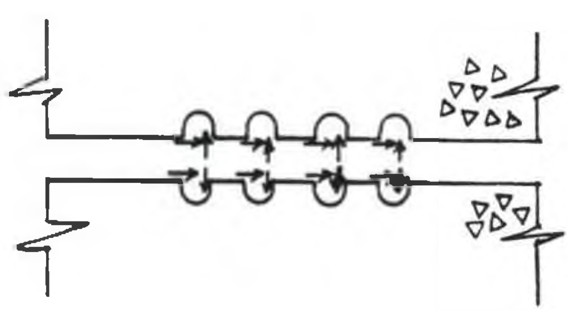
2. Concrete cover splitting failure: The forces between a deformed bar and concrete which may cause splitting can be viewed as in Figure 2. If the shear strength is high enough, which usually is the case with deformed bars, the highest of the principal tensile stress, or principal compressive stress will cause a failure in the concrete. If it is the principal tensile stress which exceeds the tensile strength of the concrete, then cracks will appear transverse to the principal tensile stresses. Principal tensile stresses are the horizontal components in Figure 2c. This transverse cracking can be viewed as in Figure 3. When the cracking occurs, the bond forces must radiate out from the perimeter of the reforcing bar.[6] The outward radiating bond forces must be resisted by the surrounding concrete



(a) On Bar



(b) On Concrete



(c) Components on Concrete

Figure 2. Deformed Bar Bond Forces.[3]

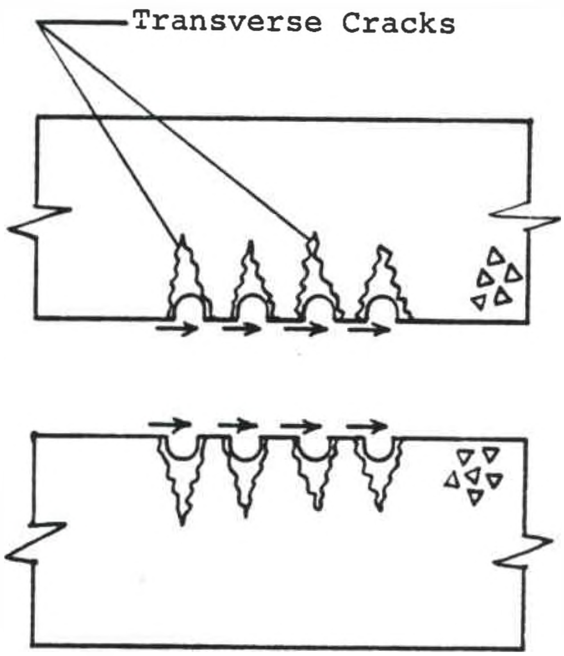


Figure 3. Transverse Cracking Due to Principal Tensile Stress.

in order to avoid sudden failure, otherwise the concrete under the splitting forces exerted by the anchored bar will split away. This type of failure is the most common in reinforced concrete structure with deformed bars.

3. Shear failure in concrete along the lugs of the bar: If the tensile strength of the concrete is high enough to resist the principal tensile stress, then bond failure will occur as shear failure along the perimeter of the bar lugs[6] (see Figure 4). In most of the practical cases, the level to achieve this type of failure is rarely reached.

Recent studies[2] have hypothesized that the action of splitting forces on concrete arises from a stress condition analogous to a concrete cylinder surrounding a reinforcing bar and acted upon by the outward radial components of the bearing forces from the bar. Those are presented as the vertical components in Figure 2c. The resisting concrete rings as seen in Figure 5, tend to balance the radial components induced by the reinforcing bar. Those rings have a tensile nature and can be called tensile stressed concrete rings. When the ring is stressed beyond its maximum capacity, rupture occurs as well as longitudinal cracks (see Figures 6 and 7). However, these longitudinal cracks may start internally and propagate outward. They cannot be seen until reaching the ultimate capacity of the concrete ring.

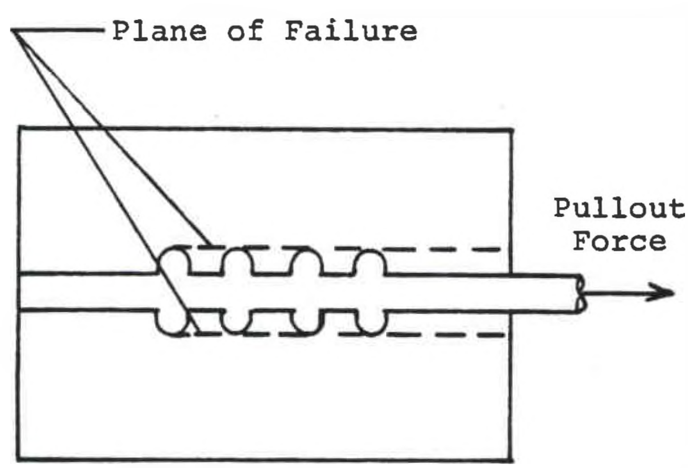


Figure 4. Shear Failure Along the Bars Lugs.

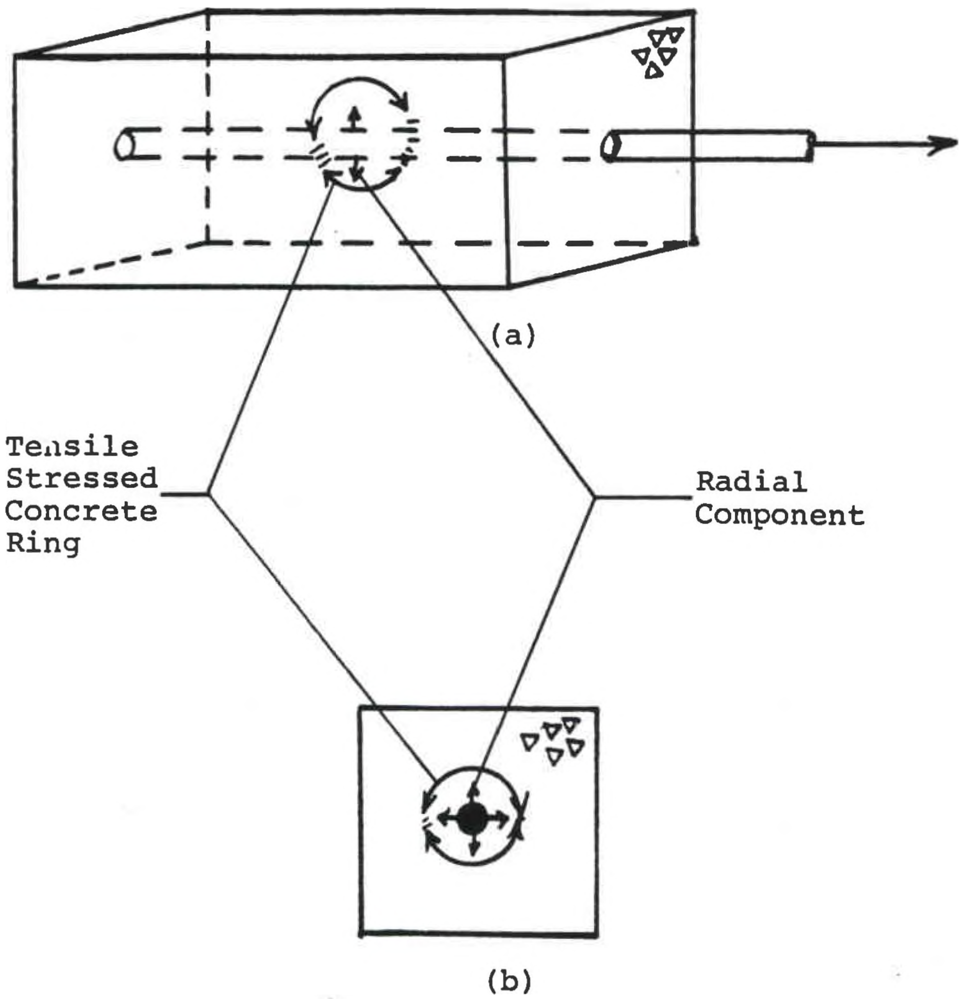


Figure 5. The Resistant Tensile Concrete Ring.

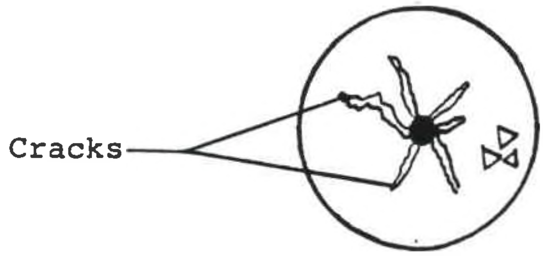


Figure 6. Longitudinal Cracks.
(Top View)

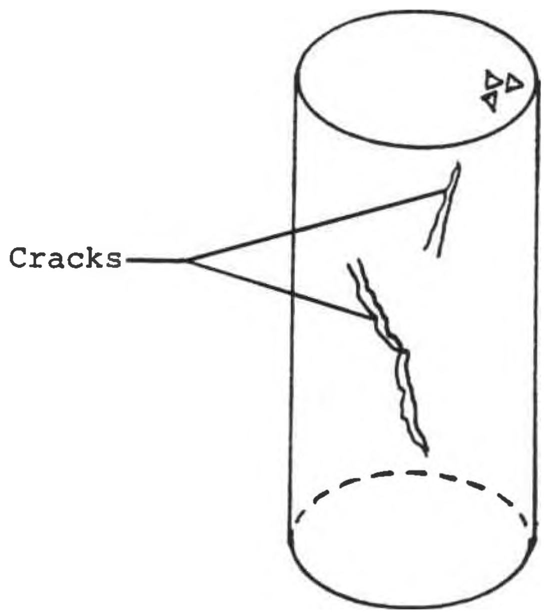


Figure 7. Longitudinal Cracks.
(Profile)

Upon the appearance of longitudinal cracks, displacement between the bar and the concrete increases considerably relative to that which existed before cracking. This displacement tends to evenly distribute bond stresses along the cracked length. The remnants of the cracked concrete ring surrounding the reinforcing bar may be thought of as cantilevers. The radial components of the anchorage force then impose a uniformly distributed load on the cantilevers (see Figure 8). When these cantilevers are stressed to their ultimate capacity, they fail according to the minimum stressed surface failure pattern.[6] This kind of failure is explosive, and normally occurs without any warning of prior ductile deformation of the concrete.

Many pullout tests have been developed to predict the distribution of bond stress. Slip of the bar relative to the concrete is measured at the bottom (loaded end) and top (free end), (see Figure 9).

Most engineers calculate bond resistance as if it is uniformly distributed over the bar embedment length. Actually, the bond stress varies greatly as slip develops along the bar. Even a very small load causes some slip, therefore, high bond stresses in the vicinity of the slip. Usually slip starts to occur at the loaded end, at which the bond stress starts to form leaving the rest of the embedment length of the bar totally unstressed as in Figure 9. The

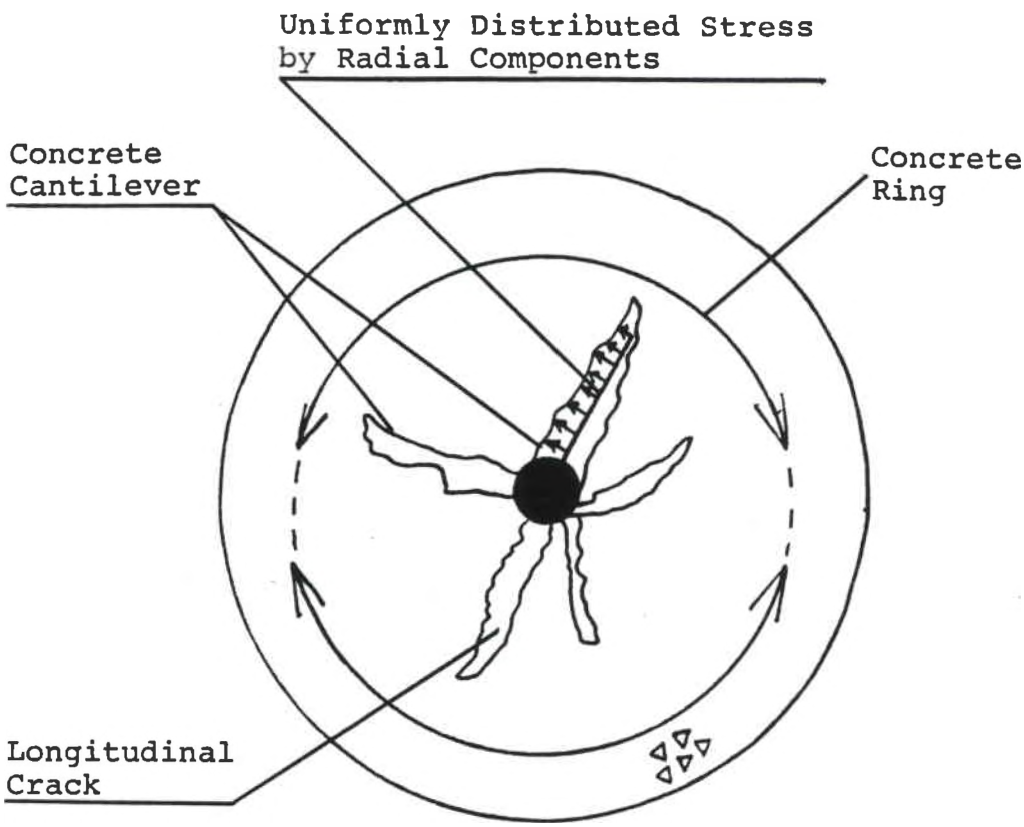


Figure 8. Even Distribution of Bond Stresses
Along the Cracked Length.

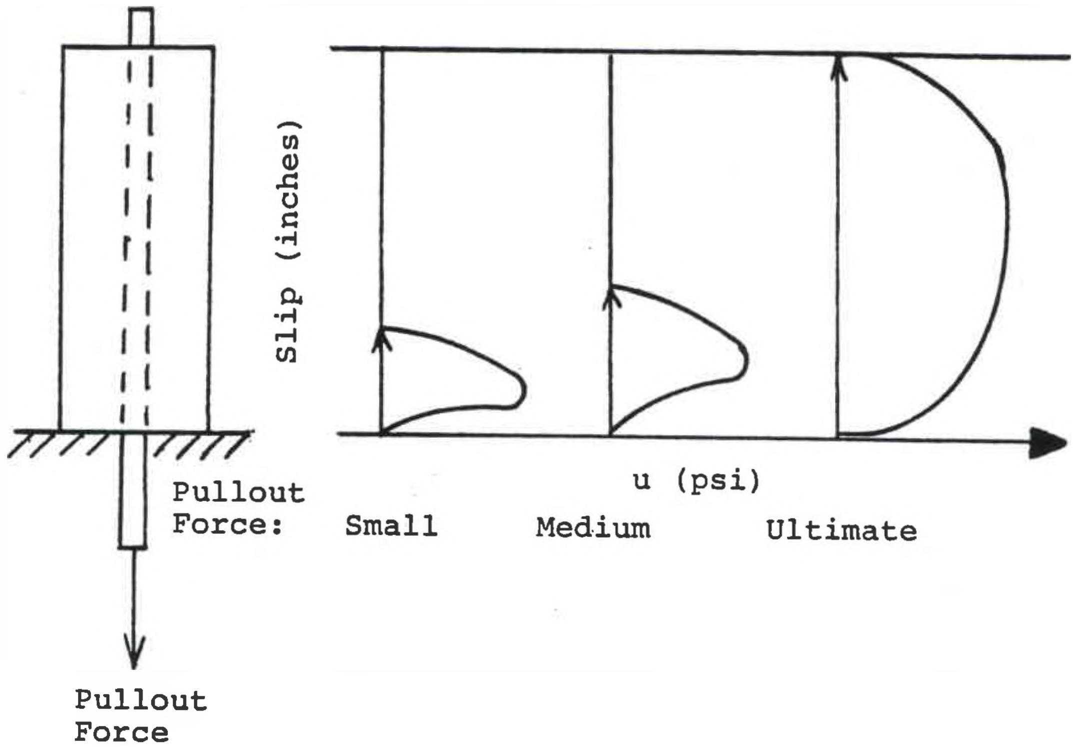


Figure 9. Slip of the Bar Versus Bond Stress (u).[3]

more the load is increased at the end of the bar, the more slip is developed, therefore the more bond stresses spread along the embedment length. Eventually, upon further application of the load, the full embedment length becomes stressed. When the slip reaches the unloaded end, the maximum resistance has been reached (bar is fully stressed). The variation of bond stress versus slip is described in Figure 9.

If bond resistance at failure is to be determined, a simplified approach depicted in Figure 10 can be applied using as embedment length, L, a pullout force, T, is applied to a bar of diameter d. The unit bond stress is denoted by the symbol u and is assumed to be uniform along L.

The average bond stress times the area over which it is acting should balance the pullout force. This can be written as follows:

$$u \cdot \pi \cdot d \cdot L = T = A_s \cdot f_s$$

where: u = bond stress in unit force per unit area

$\pi \cdot d$ = perimeter of bar in unit length

L = embedment length

A_s = area of bar in unit area

f_s = stress in steel bar in force per unit area

which yields

$$u = (A_s \cdot f_s) / \pi \cdot d \cdot L \qquad (3)$$

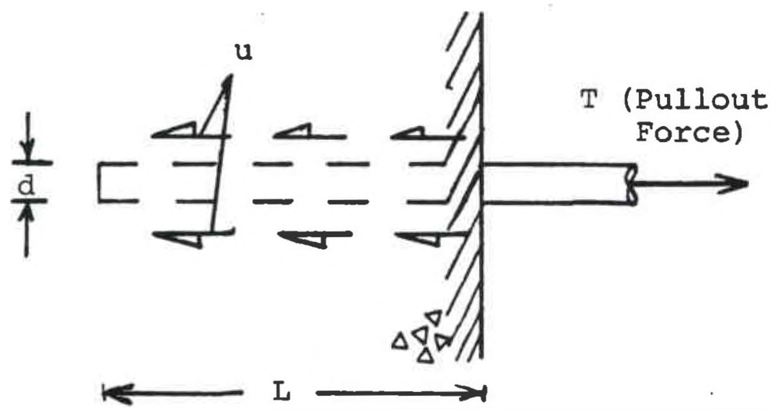


Figure 10. Uniform Bond Stress Along Embedment Length.

$$A = \pi \cdot d^2 / 4 \quad . \tag{4}$$

Substituting (4) into (3) yields:

$$u = f_s d/4L \quad . \tag{5}$$

From (5) one can observe that bond stress is directly proportional to the diameter d and inversely proportional to the embedment length L.

From the 1963 ACI building code for ultimate strength design:

$$u_{ultimate} = \frac{8.0\sqrt{f_c'}}{d} < 800 \text{ psi}$$

and from the above derived Equation 5:

$$L = \frac{f_s d}{4 \dfrac{8.0\sqrt{f_c'}}{d}}$$

or

$$L = \frac{d^2}{4} \frac{f_s}{8\sqrt{f_c'}}$$

multiplying the top and the bottom of this equation by π yields:

$$L = \frac{\pi}{\pi} \frac{d^2}{4} \frac{f_s}{8\sqrt{f_c'}} \quad .$$

But $\pi d^2/4 = A_b$ which is the area of the bar. So L becomes:

$$L = \frac{0.04 \, A_b f_s}{\sqrt{f_c'}} \tag{6}$$

which matches exactly with Equation (2) which was given in the 1983 ACI Code 12.2.2.

Bond Failure - Two-Way

As previously indicated in the literature search, a comparative study of bond failure in the one-and two-directions slabs was not found. However, speculations about the load carrying capacity in two-way reinforcing are presented in the following paragraph.

Let us analyze the load carrying capacity of the two-way reinforcing in two stages: stage one, before any cracks form in the concrete; and, stage two, after formation of cracks in the concrete.

In stage one, upon the load application on Bar A, and due to the Poisson's effect, the specimen is going to elongate in the direction of bar A and contract in the direction of bar B. The exagerated shape of the specimen after loading is shown in Figure 11, and marked with dashed lines. Consider the two concrete elements in the vicinity of Bar B which are marked by dots 1 and 2. Those 2 concrete elements are going to approach each other because of the Poisson's effect mentioned above. By getting those particles close together the mass of particles per unit volume of concrete is going to get denser, therefore, more grip on bar B and more shear resistance is created along that bar. By creating higher shear resistance along bar B, this bar

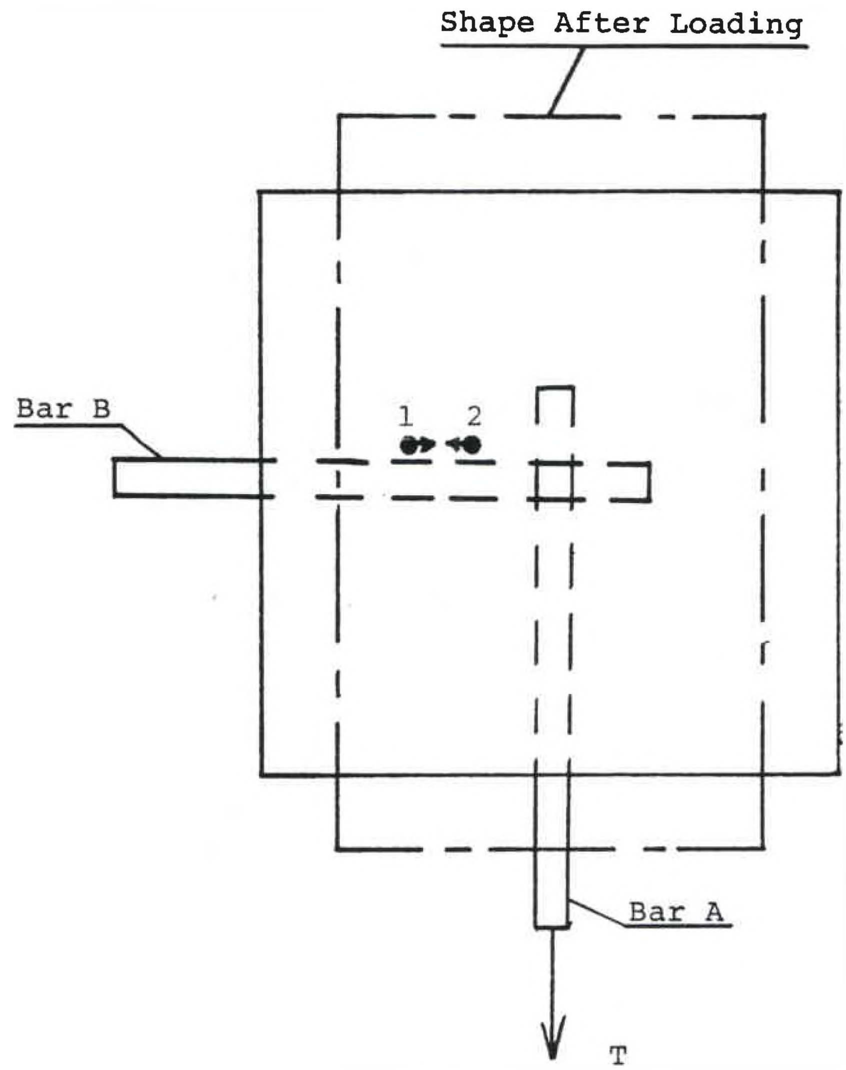


Figure 11. Exagerated Shape of Specimen After Loading.

will support higher pullout load. The above qualitative analysis leads us to say that, applying a load on Bar A will increase the bond resistance along bar B.

One more thing can be mentioned, bar B may act as a stirrup to prevent shearing of concrete and crack formation by application of load on bar A. Bar A can be viewed as being surrounded by a concrete cylinder that tends to shear off along the dashed lines in Figure 12. Since Bar B is placed normal to the line of action of the pullout force exerted on Bar A, it will provide a tendency to prevent this shearing action from happening. The same argument can be applied to the other bar. However, we are still in stage one which assumes no crack formation.

In stage 2, cracks start to form in the vicinity of the steel bars. One might reason that unit bonding capacity of two-way reinforcement might be reduced in stage 2 by the influence of the progressive failure incurred in each direction. Because cracks start to form internally around the bars upon the application of the load, gaps are created which can be expected to increase gradually with increasing bond stress (see Figures 11 and 12). The propagation of an increasing number of cracks thru the concrete medium will gradually influence its bond or grip on bars which lie perpendicular to one another and permit these bars to slide more freely thru the formed gaps. The decreased bond

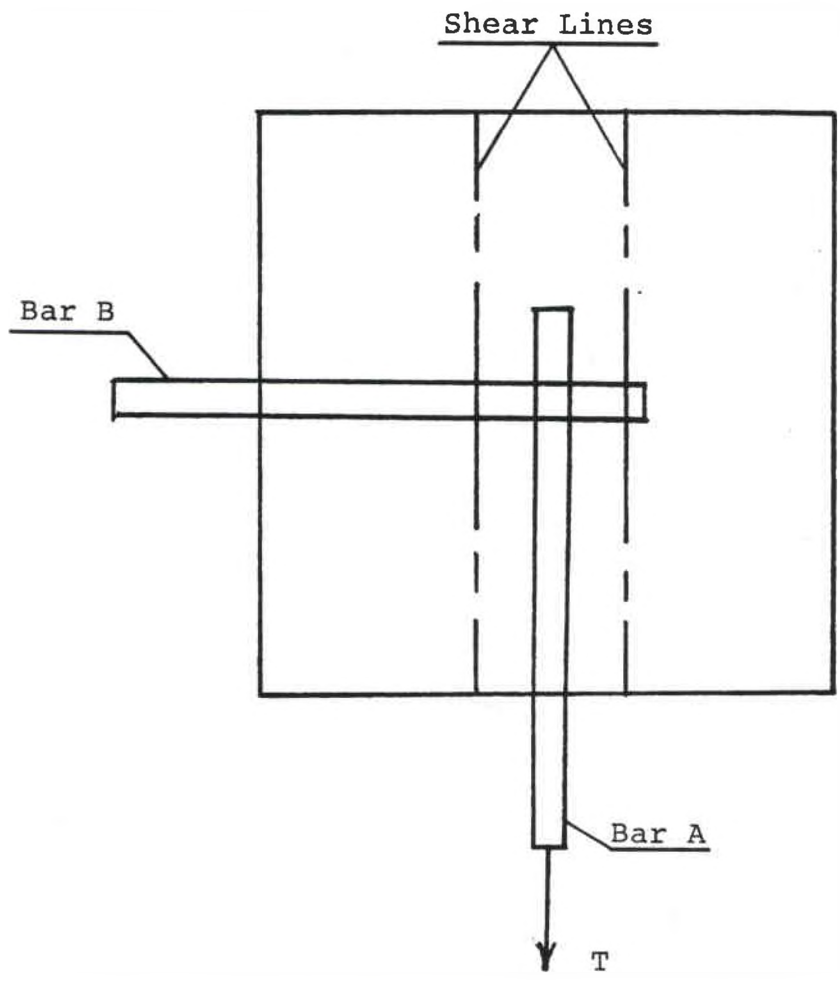


Figure 12. Bar B Opposing Shearing of the Concrete Caused by Pullout Force (T) on Bar A.

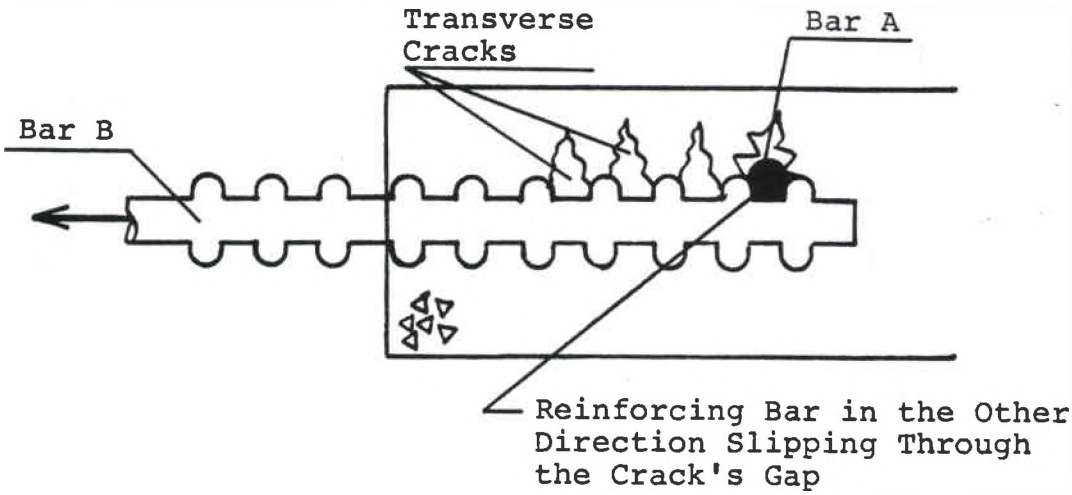


Figure 13. Formed Cracks Reducing the Reinforcing Capacity in Two-Way Slab. (Side View)

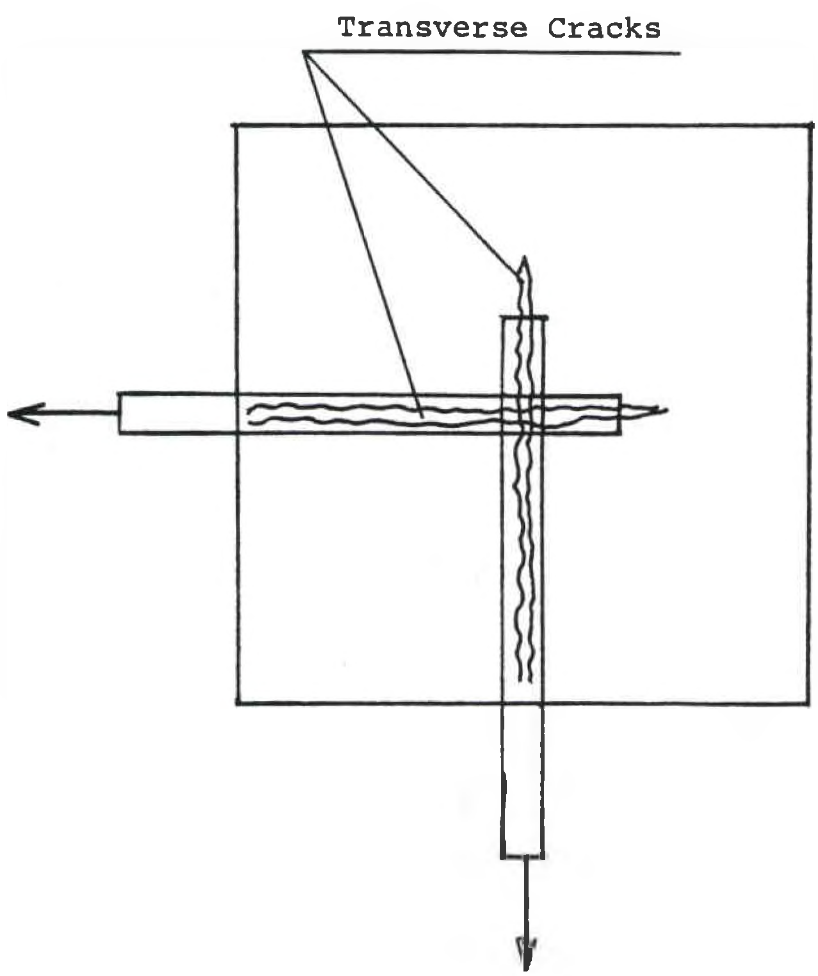


Figure 14. Transverse Cracks in Two-Way Slab. (Top View).

between steel and concrete resulting in the two-directional case will reduce the overall reinforcing capacity in each direction to below that which might be expected in the one-directional case.

It should be noted that reduced bond capacity in stage 2 caused by load application to either or both reinforcing bars in the two-directional case <u>does not</u> necessarily imply that single direction reinforcing is preferable. Two-way design still contributes much greater structural capability than the one-way design.

CHAPTER III

TESTING

Preliminary Mix

A preliminary mix study was conducted to determine the proper proportions of ingredients to achieve 3000 psi compressive strength at seven days. Eight cylinders having 4" diameter and 8" length were cast. Testing was done on a 300,000 lb. Riehle compression test machine. The results of the study are shown in Table 1.

The followings are the proportions that were used in the mix:

- Cement = 481.7 #/yd^3
- Coarse aggregate = 1732.4 #/yd^3
- Fine aggregate = 1329.5 #/yd^3
- Water = 341.5 #/yd^3

One-Way Test

Introduction

One cylinder made of two 6" x 12" cylinders was cast to demonstrate the nature of bond failure. Also three one-way square specimens with different embedment lengths 12", 14", 16" were made. These specimens were 24" x 24" x 6", note

TABLE 1

COMPRESSION TEST - PRELIMINARY MIX STUDY

Portland Cement Type III (Early Strength Cement)

Curing Time (Days)	Force (Pounds)	Cylinder Data			f_c' (psi)	Average Strength (psi)
		Diam. (in.)	Length (in.)	Area (in.)		
4	41,500 42,000 44,000	4 4 4	8 8 8	12,567 12,567 12,567	3302.46 3342.25 3501.41	3380
7	35,000 45,000 38,000	4 4 4	8 8 8	12,567 12,567 12,567	2785.21 3580.99 3023.94	3130
14	45,000 43,000	4 4	8 8	12,567 12,567	3580.99 3421.83	3500

that all references to square specimens applies to those whose corners were trimmed to remove extraneous concrete and thereby reduce the specimen's total weight, (see Figures 15 and 21). The concrete was proportioned to give a compressive strength of 3000 psi after seven days. One #5 bar with a yield strength of 40,000 psi was embedded in each of the cylinder and square specimens. Along with each of the cylinder and the three square specimens, one 6" x 12" cylinder was made from the same mix to measure the actual compressive strength of each of the specimens at the time of testing.

Technique

All the one-way tests were conducted in the universal testing machine in the materials testing laboratory of the Department of Civil Engineering at the University of Dayton. The general view of a typical test set-up is shown in the photograph, designated Figure 16. The 6" x 12" cylinders that go along with each of the specimens were tested with the 300,000# Riehle machine in the concrete laboratory at the University of Dayton.

As the test proceeded, the upper head of the testing machine moved upward bearing against the concrete, while the reinforcing was held fixed by the grip jaws of the fixed lower head (see Figures 17 and 18). As the pullout load continued to increase a considerable flaking of the bar was

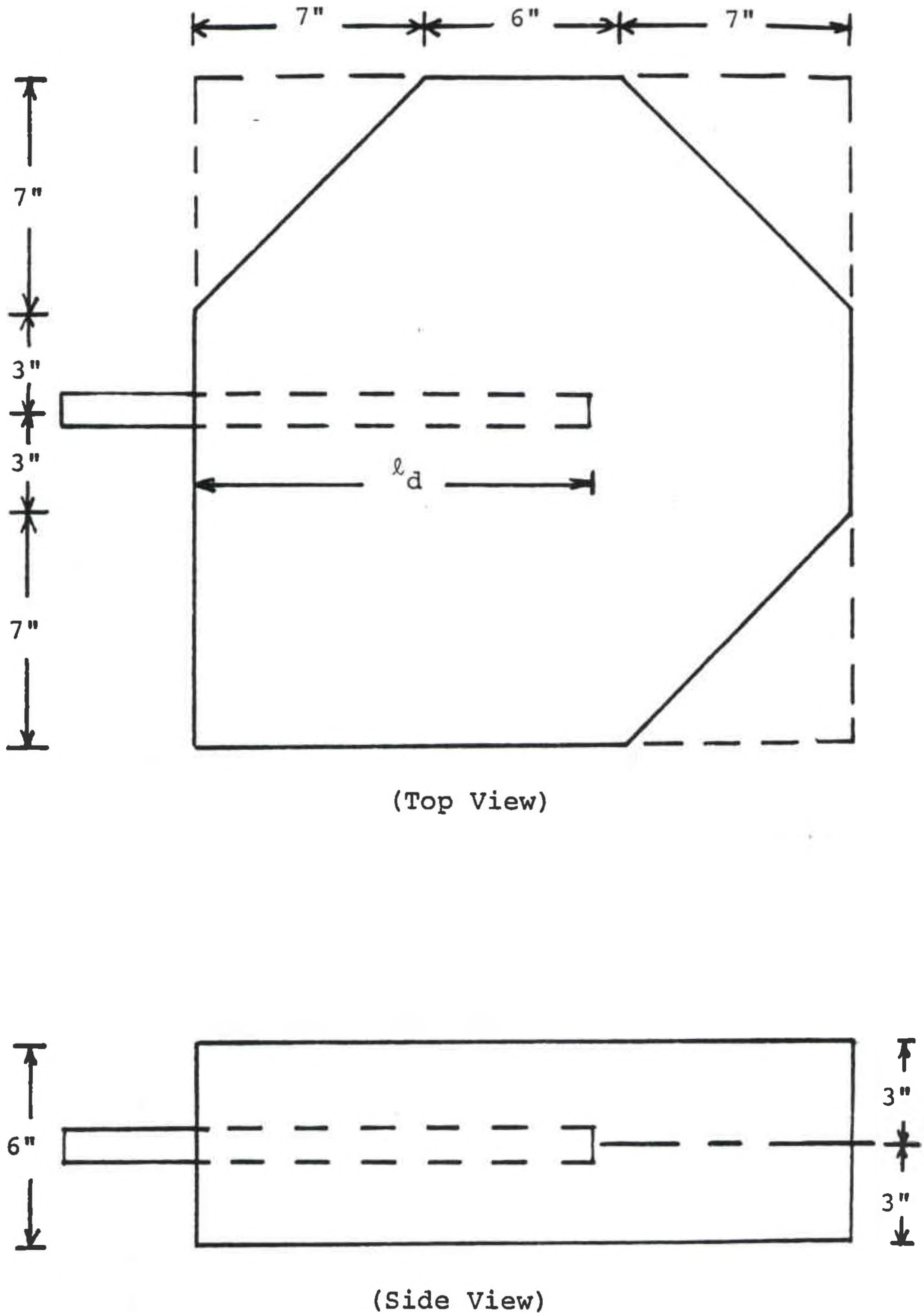


Figure 15. One-Way Specimen.

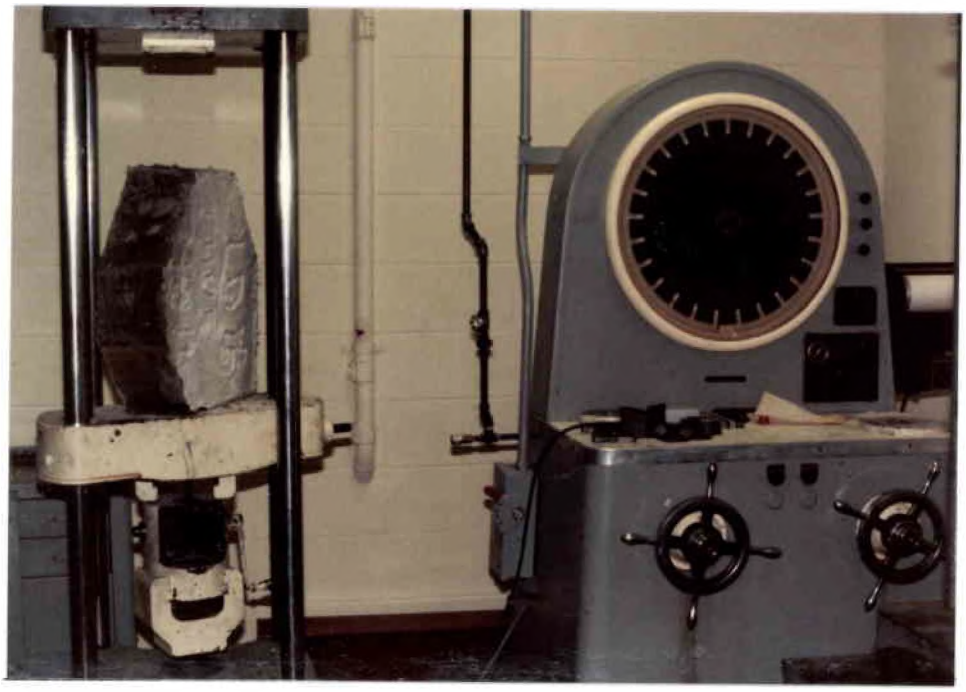


Figure 16. General View of the Loading Device and Typical Test Set-up.

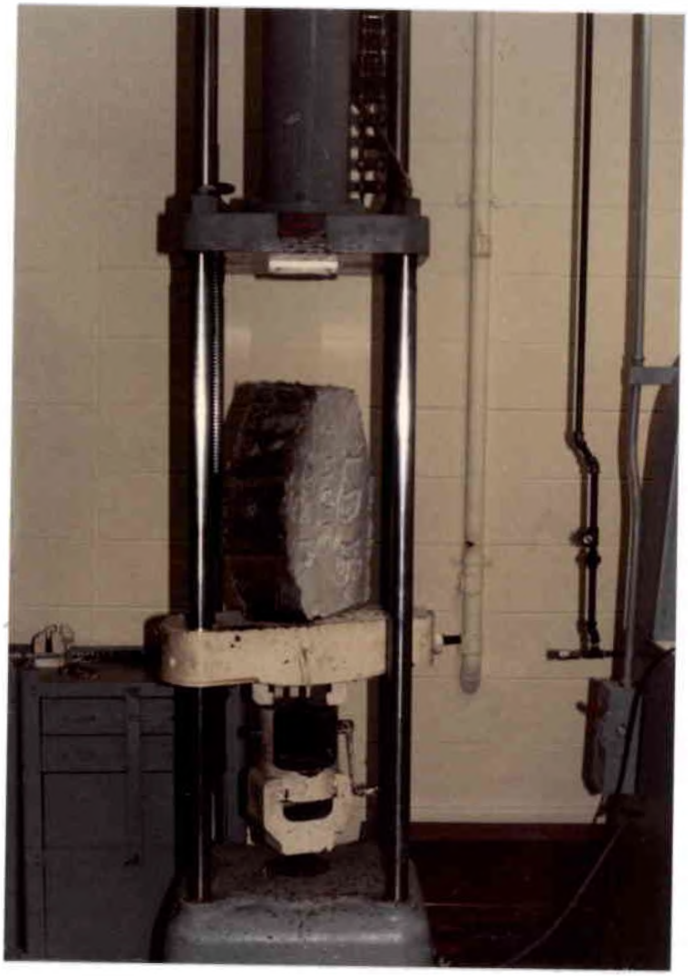


Figure 17. General View of the Upper Moving Head and Lower Fixed Head.

Figure 18. Close View of the Grip Jaws Fixing the Bar.

evident. In the range of 14,400 pounds the speed of the load indicator dial slowed down relative to its previous speed indicating a yield of the steel. This behavior was also encountered for all three square specimens. After yielding, the load continued to increase again leaving the elastic range, heading toward the plastic range. The load was increased until the load indicator dial moved back sharply indicating a pullout of the reinforcing bar. Readings were recorded at the yield point and at pullout. In the case of the reinforced cylinder a failure of the concrete was observed (see Figures 19 and 20) due to the action of splitting forces on the concrete as explained in Chapter I.

The results for the three square specimens are summarized in Table 2.

Two-Way Test

Introduction

Three two-way square specimens with different embedment lengths of 12", 14", 16" were made. The concrete was designed to reach a compressive strength of 3000 psi after seven days. Two #5 bars, with a yield strength of 40,000 psi were embedded perpendicular to one another in each of the square specimens (see Figure 21). Along with each of the three square specimens one 6" x 12" cylinder was made from the same mix to measure the actual compressive strength of each of the specimens at the time of testing.

Figure 19. Longitudinal Cracks in a Cylinder Specimen. (Profile)

Figure 20. Longitudinal Cracks in a Cylinder. (Top View)

TABLE 2

ONE-WAY SLAB TEST RESULTS

Embedment Length (inches)	Yield Occurred at (Pounds)	Pullout Load (Pounds)	Compressive Strength (psi) at Time of Test
12	14,000	22,500	2,300
14	14,240	22,700	3,183
16	14,400	23,500	2,500

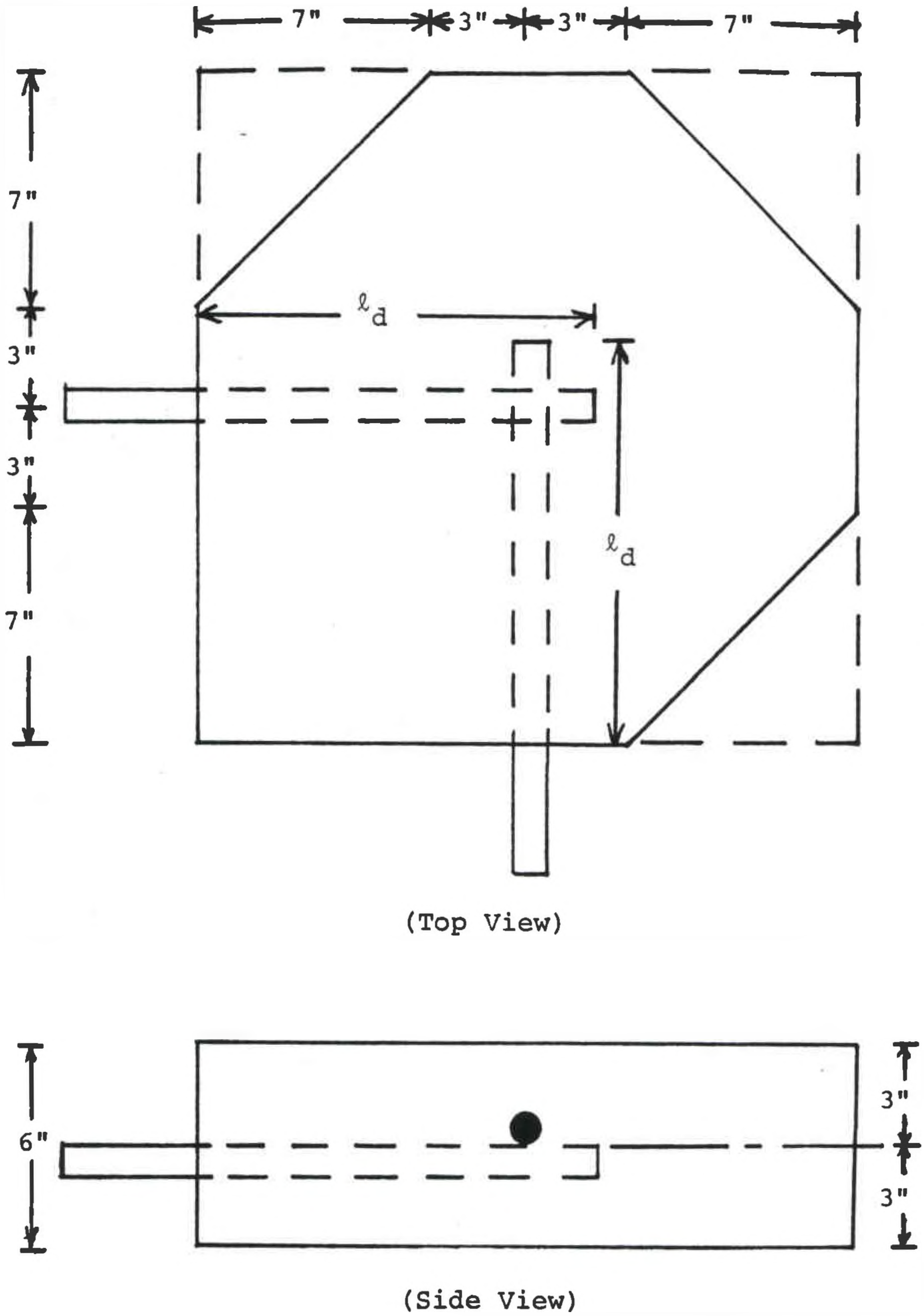


Figure 21. Two-Way Specimen.

The lack of a standard testing procedure to perform the two-way test required the design of a new test set-up which provided, simultaneously, a pullout force in two directions. A general view of a typical test set-up is shown in the photograph, designated Figure 22.

Technique

Starting from the slab face and proceeding outward along the length of the bar, the test set-up shown in Figures 23 and 24 was assembled as follows. An aluminum plate was grouted to the concrete to act as a bearing plate between the applied force and the concrete face of the slab. This plate had a hole which was big enough for the bar to go through it. Next a hollow high strength bolt with nut was slid all the way back toward the aluminum bearing plate. The nut was all the way on the bolt prior to the start of the test. Another aluminum plate with a hole in it was placed after the bolt. An aluminum load cell which has a hollow cylindrical shape was placed next. It has a larger inner diameter than the reinforcing bar (#5 bar) therefore some paper tape was wrapped around the bar to help center the load cell. More details about the load cell are given later in this chapter. Finally, a chuck of the type used for gripping pretension strands in prestressed concrete was installed to grip the reinforcing bar. This sequence of set-up was the same for both bars in the two directions.

Figure 22. A General View of a Typical Two-Way Test Set-up.

Figure 23. Two-Way Test Set-up.
(Side View)

Figure 24. Two-Way Test Set-up.
(Top View)

Two $2\frac{1}{2}$" size wrenches were needed. One wrench to tighten the nut, the second to prevent the bolt from rotating. Upon tightening the nut, the bolt will push away from the slab toward the load cell which is prevented from slipping by the gripping chuck placed after it. While the bolt is moving toward the cell the steel bar is elongated, therefore a force which is almost purely axial is induced in the bar. The intensity of the induced force is detected by the load cell. This procedure was done simultaneously on both bars by applying small increments of loads, with special attention given to keeping the loads as nearly equal as possible.

The aluminum load cell has a hollow cylindrical shape. Two strain gages were installed adjacent to each other at mid-height on the outer surface of the cell. Figure 25 shows the location of the strain gages. The two strain gages were installed with the assistance of professor E. H. Payne. Locating the strain gages was based on the following.

The load was assumed to be applied purely axial. Nevertheless, in case of occurrence of irregularities in the test set-up causing a non-axial load, or any accidental bending, the chosen location of the strain gages will provide purely axial load data. The strains in the load cells were measured with a 24 channel commercial strain indicator.

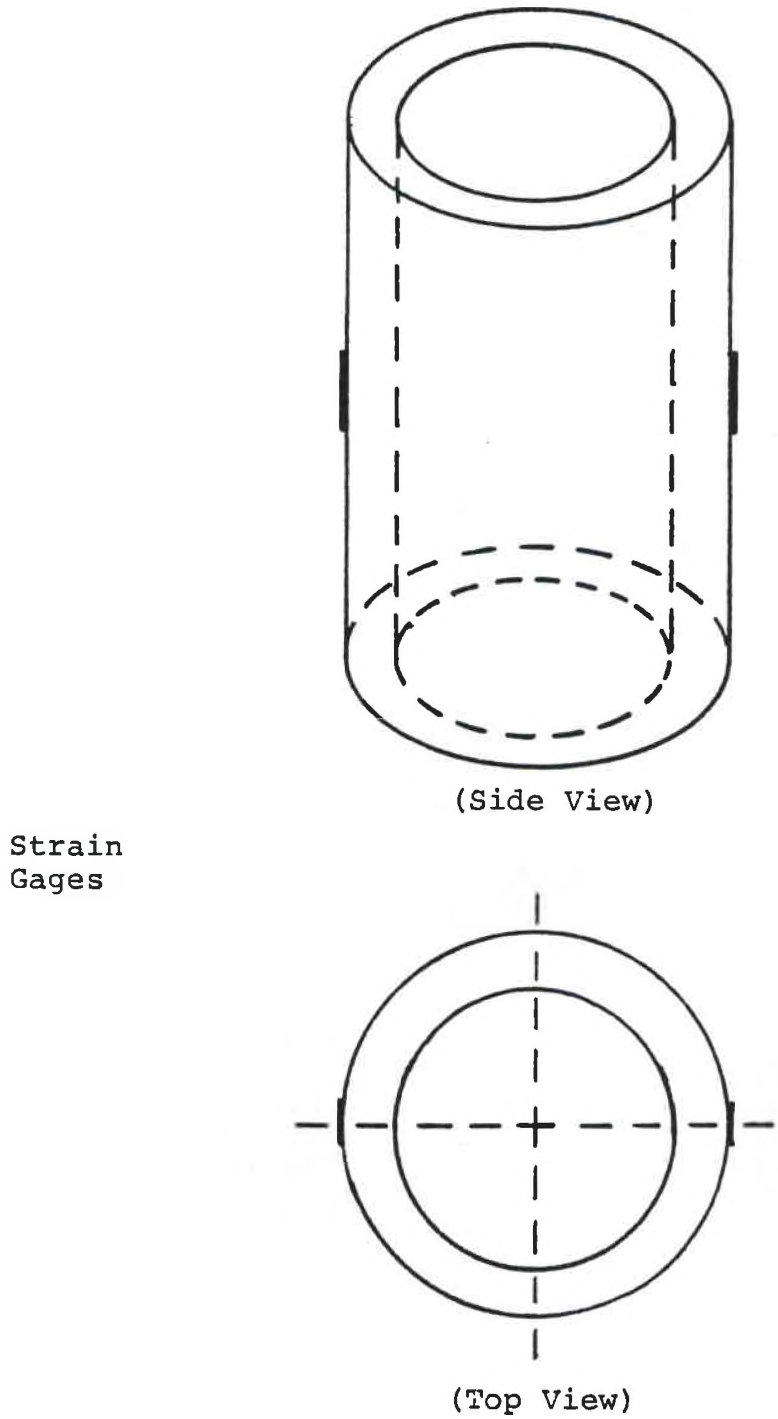


Figure 25. Strain Gages Location on Load Cell.

The load cells were numbered 1 and 2. Wires from cell #1 and cell #2 were connected to channels 1 and 2 respectively (see Figure 22). Those wires were taped to the floor for safety precautions (see Figure 23).

Since the load cells allow us to read strain readings, a calibration of those cells was needed to convert strain data to load data. The Baldwin Compression machine was used for this purpose. The load cells were subjected to compressive load, one at a time.

A load at 1000# was applied at first, then 2000#. After that a load increment of 2000# was applied. Strain readings were taken after each loading application. Two sets of calibration data were taken for each cell. One set under loading condition and the other under unloading condition as shown in Tables 3 and 4.

A programmable calculator was used to find the line of best fit for the strain data obtained by the calibration process. The equations for the best fit are also included in Tables 3 and 4. The loading process was done simultaneously on both bars applying small increments of loads at a time as shown in Table 5. Care was taken to keep loads of nearly matching values in both bars. Application of the load continued until rupture of one of the bars occurred (See Figures 26 and 27). Due to the elongation of the bars under the load, which was more than the thickness of the

TABLE 3

CALIBRATION OF LOAD CELL #1

LOAD x 1000 (Pounds)	Strain ($x10^{-6}$ inch/inch)	
	Loading Condition	Unloading Condition
0	0	0
1	80	60
2	150	150
4	290	290
6	430	430
8	570	570
10	710	710
12	850	850
14	990	990
16	1130	1130
18	1270	1270
20	1420	1410
22	1560	1550
24	1700	1680
26	1840	1820
28	1970	1960
30	2110	

$y = (m)x + b$

Load = (Slope)(Strain Reading) + (y intercept)

Load = (0.0142534013)(Strain Reading) + (0.1016761627)

TABLE 4

CALIBRATION OF LOAD CELL #2

LOAD x 1000 (Pounds)	Strain ($x10^{-6}$ inch/inch)	
	Loading Condition	Unloading Condition
0	0	0
1	80	80
2	150	1601
4	290	310
6	430	450
8	570	600
10	730	740
12	870	880
14	1010	1030
16	1150	1170
18	1300	1320
20	1450	1460
22	1590	1600
24	1730	1740
26	1880	1880
28	2000	2010
30	2150	

$$y = (m)x + b$$

Load = (Slope)(Strain Reading) + (y intercept)

Load = (0.013943112)(Strain Reading) + (0.171046996)

TABLE 5

LOADING STRAIN DATA (x 10^{-6} inch/inch)

LOAD CELL #1	LOAD CELL #2
0	0
70	70
140	200
260	270
385	400
530	540
670	690
715	800
800	800
830	830
885	885
880	895
890	920
910	950
910	970
915	990
915	1,000
910	1,000
980	1,000
1,000	1,000
1,015	1,050
1,045	1,080
1,065	1,110
1,100	1,140
1,115	1,180
1,150	1,230
1,190	1,280
1,230	1,310
1,280	1,295
1,295	1,325
1,320	1,330
1,340	1,340
1,345	1,350
1,365	1,365
2,000	1,400
1,410	1,410
1,420	1,500
1,455	1,455
1,480	1,480
1,480	1,500
1,490	1,530
1,510	1,560
1,510*	1,580*

TABLE 5 (Continued)

LOADING STRAIN DATA (x 10^{-6} inch/inch)

LOAD CELL #1	LOAD CELL #2
Start @ 0	Start @ 0
780	950
1,010	1,030
1,160	1,200
1,350	1,350
1,520	1,500
1,560	1,560
1,590	1,620
1,600*	1,620
Start @ 0	1,590
1,450	1,590
1,550	1,550
1,450	1,660
1,410	
Rupture $\boxed{1,600}$	
Rupture Load = 22703.76#	

*Unload and start again

Figure 26. Bar Rupture. (Side View)

Figure 27. Bar Rupture. (Top View)

nut, we had to unload, shift everything toward the slab, and load again. When this is encountered it is mentioned in Table 5.

The results of the two-way test are summarized in Table 5. Only one two-way test was performed on the specimen with 12" embedment length. Details why only one two-way test was performed are in the following discussion.

Discussion

Since this work is experimental, some error associated with the results is expected. These errors might occur anywhere from casting to testing. Quantifying those errors was very hard because of the lack of a perfect model to compare with and because those errors were relatively small. As seen from the compression tests in Tables 1 and 2, the compressive strength varies from one specimen to another. Since the specimens have different strengths, one might not expect to get exactly the same test results. The workmanship might also contribute some error to the test. A slight error might be encountered in setting the reinforcing steel to the proper embedment length. Also, trying to place the steel bars horizontally might be considered as a source of error. One more thing which may be mentioned about the workmanship of the specimens is, when jiggling or vibrating the concrete, the test cylinder that goes along with the specimen to predict its compressive strength, may not have been vibrated as much as the specimen itself. That suggests that our compressive concrete strength data may not be accurate. Another error contributing factor was taking readings from the dial gage of the Universal testing machine. This machine was only used to test the one-way specimens. Difficulty was encountered in taking readings as it was hard to define when the steel reached its yield point. Taking readings from the readout equipment of the strain gages in the two-way test had to contribute some error. In taking those readings the least count was 10; Table 3 has a least count of 10 in 2110 or 1 out of 211, which is about 1/2% error. The testing machine has a ±1% error which accumulate the error associated with those readings to 1.5%.

Being practical, specimens were chosen to have square shapes because the two-way method of reinforcing is commonly used in square shaped slabs. If a retangular slab is reinforced in the two directions, the steel in the long direction will support a very small portion of the applied load compared to that supported by the steel in the short direction.

In the specimens a minimum embedment length of 12 inches and a concrete compressive strength of 3000 psi were used. Those numbers were chosen based on the ACI building code requirements for reinforced concrete, namely ACI 12.2.2 and ACI 12.2.5.

According to those specifications the embedment length ℓ_d should be:

$$\ell_d = \frac{0.04}{\sqrt{f_c'}} A_b f_s \qquad (7)$$

or

$$T = A_b f_s = \frac{\ell_d \sqrt{f_c'}}{0.04} \qquad (8)$$

where A_b is the area of the steel bar in square inches. The pullout load T is directly proportional to the development length ℓ_d and the $\sqrt{f_c'}$. In order to make the steel pullout at a lower load, one should lower ℓ_d and f_c'.

From ACI 12.2.5 ℓ_d cannot be less than 12". The lowest concrete compressive strength that is used in the United States is 3000 psi so the expected development length will be:

$$\ell_d(\text{required}) = \frac{0.04(0.31)(75{,}806)}{\sqrt{3000}} = 17.16" \qquad (9)$$

where 75,806 = f_s, it was computed as follows:

$$f_s = \frac{\text{Rupture load}}{\text{area of the bar}} = \frac{23{,}500 \text{ lbs}}{0.31 \text{ in}^2} = 75{,}806 \text{ psi}$$

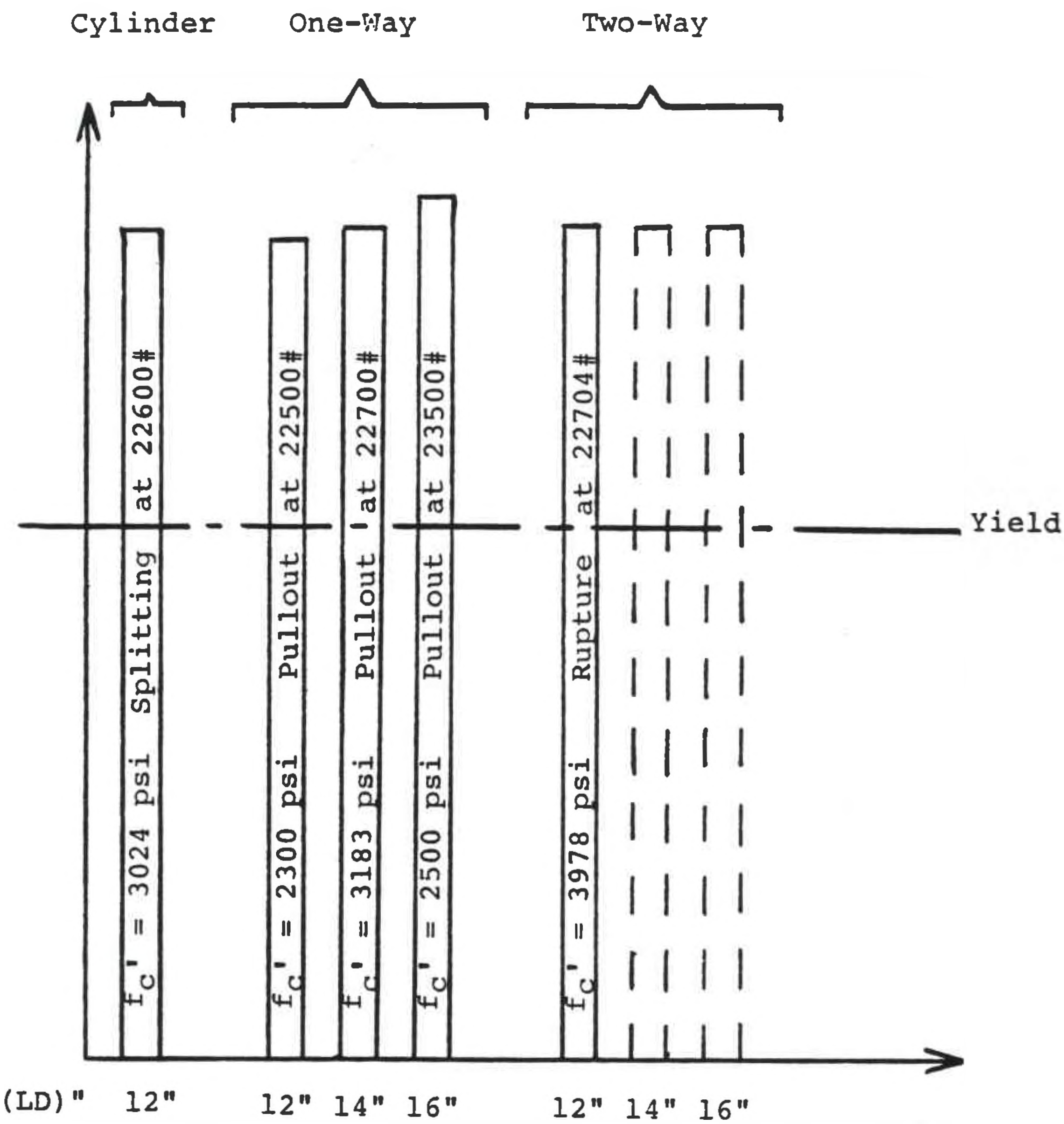
Supportive evidence to the nature of bond failure, explained earlier in Chapter II, was obtained. The cylinder pullout test showed longitudinal cracks due to the action of splitting forces (see Figures 19 and 20). A sledge hammer was used to break the concrete around the reinforcing bar. It was observed that the lugs in the concrete near the loaded end were not as deep as they were near the other end.

This supports the theory of the distribution of bond stresses mentioned earlier in Chapter II, (see Figure 9). From this we conclude that there was some slippage of the steel bar near the loaded end which eroded the lugs near that end. Furthermore, bond stress had developed in the vicinity of the loaded end when slippage had occurred.

Looking at Table 6 we observe that the concrete cylinder has experienced a bond failure by splitting of the concrete, while the one-way square specimen encountered pullout of the bar at almost the same load intensity even though they both have 12" embedment length. This concrete failure can be referred to the action of splitting forces on concrete explained in Chapter II. In the case of the cylinder, the resisting concrete ring around the bar has been stressed beyond its maximum capacity which caused the splitting of the concrete cylinder. In the case of the one-way square specimen, a pullout of the bar occurred instead. This is because there was more concrete mass surrounding the bar which produced a stronger concrete ring capacity, therefore splitting of the concrete did not occur. One more point to be mentioned is, the compressive strength of the concrete cylinder was 3024 psi which is higher than for the one-way square specimen which was 2300 psi. This high difference in the concrete compressive strength did not prevent the cylinder from splitting, which draws our attention more and

TABLE 6

BAR CHART RESULTS SUMMARY



Cylinder: Bond Failure Splitting of Concrete

One-Way: Pullout

Two-Way: Bar Rupture

more to the importance of the concrete mass that surrounds the reinforcing bar as was provided in the one-way square specimen.

In the one-way test, yield occurred at 14000#, 14240#, and 14400# while pullout occurred at 22500#, 22700#, and 23500#. The specimen's concrete compressive strengths were 2500 psi, 3183 psi, and 2500 psi for embedment lengths 12", 14", and 16" simultaneously, (see Tables 2 and 6).
For the two-way test, yield was very hard to detect but was felt to be in the range between 14000# and 15000#. No pullout occurred, instead a rupture of one of the two reinforcing bars occurred, (see Figures 26 and 27). This was encountered at a load value of 22704#. This specimen had a 12" embedment length and a concrete compressive strength of 3978 psi, (see Table 5). No further tests were conducted on the other two two-way specimens. They had 14" and 16" embedment lengths. Because no pullout of the bars had occurred for the 12" embedment length specimen, one can easily expect that pullout will not occur for an embedment length longer than 12". This is represented by dotted lines in Table 6.

When comparing the results obtained from both tests, the one-and two-way tests, one has to take into consideration two factors, the embedment length and the concrete compressive strength. Comparing the one- and two-way specimens that had 12" embedment length, one can observe that the steel had gone beyond its yield strength in both specimens. Furthermore, the two-way specimen experienced a rupture of one of its reinforcing bars at 22704# while the one-way specimen encountered a pullout of its reinforcing bar at 22500#. Nevertheless, the fact that both specimens don't have the same concrete compressive strength should not be disregarded. The two-way specimen had a much higher strength of almost 4000 psi compared to 2300 psi for the one-way specimen.

Trying to justify the mode failures of our specimens, let us run a numerical analysis based on equation (9) which states:

$$\ell_d(\text{required}) = \frac{0.04\ A_b f_s}{\sqrt{f_c'}}$$

looking at the one-way specimens that have 12" embedment length:

$$\ell_d(\text{required}) = \frac{0.04(0.31)(75{,}806)}{\sqrt{2300}} = 19.60"$$

Since we only have 12" embedment length as compared to 19.60", pullout of the bar was encountered.

Using the same formula for the 14" and 16" one-way specimens, we find that ℓ_d(required) are 16.66" and 18.80" respectively and in all cases the required embedment lengths are higher than the provided ones which led to pullout of the bars.

In the case of the two-way specimen that has 12" embedment length, one of its bars encountered rupture at 22704#. According to the speculations about the mechanics of bond failure in two-way slabs which was mentioned in Chapter II, one can say that the specimen stayed in stage 1 and never reached stage 2. That is, there was no crack formation in the concrete prior to rupture. The Poisson's effect increased the bond resistance along the bars in the two directions which did not allow a pullout but brought the steel to rupture instead. If a higher strength steel were used, a possibility of reaching stage 2 exists. Upon formation of cracks the grip of concrete on the steel will decrease which will speed up the process of failure.

CHAPTER IV

CONCLUSIONS AND RECOMMENDATIONS

This thesis is a limited study of the effect of two-way reinforcing on bond stresses. The limitations of this experimental work are:

- The concrete compression strength was limited to 3000 psi.
- The bar size was limited to #5 bars.
- The cement used was limited to high early strength cement.
- The embedment length was limited to 12".
- The study was limited to five tests.

For practical purposes, many assumptions were made in the course of the study. Some of those assumptions were:

- The concrete medium was assumed to have a homogeneous nature.
- Based on the ACI code, bond stresses were assumed to be uniformly distributed along the reinforcing bar.

This great number of limitations and assumptions implies that any application of the theories discussed to practical designs should be carefully considered and probably avoided until more substantiating evidence arises.

Results of the one-way tests suggest that the specifications shown in the 1983 ACI code regarding the one-way reinforcing are adequately, safe for design purposes.

As the principal conclusion of this study, one can say that more extensive research is recommended to follow this preliminary one because, based on our results, one cannot make a general sweeping conclusion of how the two-way reinforcing behaves as compared to the one-way reinforcing.

More in-depth investigation is highly recommended with regard to the behavior of two-way bond stress. It is recommended to study the variation of bond stresses along the two reinforcing bars in the two-way method. A suggestion of how to go about doing that is, to place strain gages along the two bars, or cut the bars in half and mill channels through them, to place strain gages in those channels and weld the steel back together then embed them in the concrete (see Figure 28). This is to measure stresses and strains induced in and along the steel bars. This recommended suggestion allows one to study the interaction between the crossing bars which will lead one to be able to quantify the effect of two-way reinforcing.

Another recommendation will be, to somehow get into the concrete and find out at what stage of loading and how cracks are going to form. This may lead to a theory

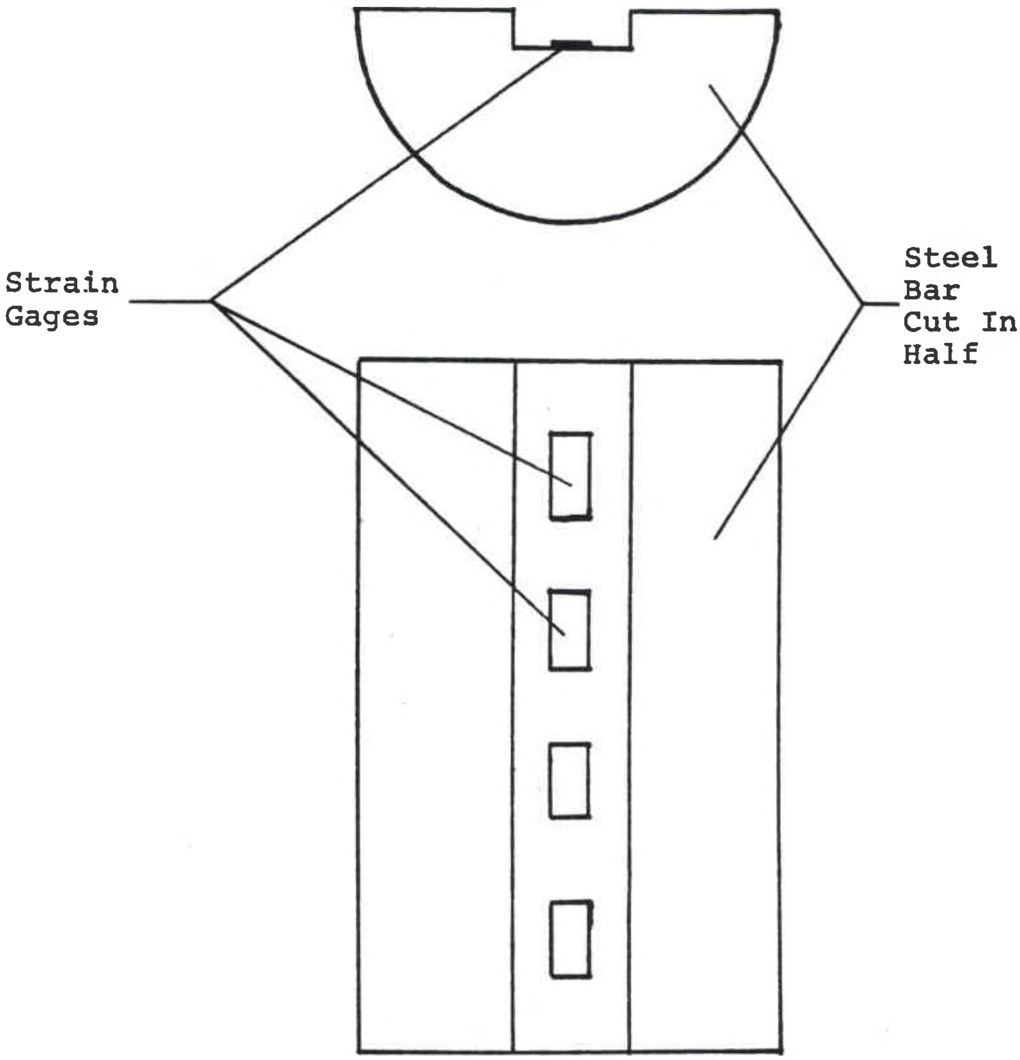


Figure 28. Bar Cut in Half Showing Strain
Gages Placed in Channel.

explaining the influence of cracks on the reinforcing. This might be achieved by subjecting the specimen under test to one of the non-destructive testing techniques such as x-ray radiography, that are used to detect internal material defects. This might provide a clearer understanding of how the concrete surrounding the bars will behave under loading.

Furthermore, it is recommended that one use shorter embedment lengths, various bar sizes and strengths, different concrete strengths, and conduct as many tests as possible. Also, the one- and two-way specimens should be cast from the same batch and tested at the same time. Bars should be cut from the same rod. One last thing, improving the two-way test set-up is recommended. This can be done by connecting two high capacity hydraulic jacks in such a way so as to provide an equal axial pullout force in the two directions at the same time. It is recommended that those jacks have a sensitive load recording mechanism to minimize the error associated with load readings.

REFERENCES

1. Boris W. Boguslavsky, Design of Reinforced Concrete (New York: The MacMillan Company), pp. 157-159.

2. Chu-Kia Wang and Charles G. Salmon, Reinforced Concrete Design, Third Edition, (New York: Harper & Row, Publishers), pp. 185-186.

3. Phil M. Ferguson, Reinforced Concrete Fundamentals With Emphasis on Ultimate Strength, (New York: The MacMillan Company), pp. 123-129

4. Skorobogatov, S. M., "The Influence of the Geometry of Deformed Steel Bars on Their Bond Strength in Concrete." Soc. Proc. Inst. Civ. Eng, Vol. 67, Pt. 2, London, June 1979, pp 327-339.

5. Somayajim S., and S. P. Shah, "Bond Stress Versus Slip Relationship and Cracking Response of Tension Members." American Concrete Institute/Journal, Vol. 78, No. 3 May, 1981, pp. 217-225.

6. Ralejs Tepfers, "Lapped Tensile Reinforcement Splices," Amer. Soc. of Civil Engs., Journal of the Structural Division, V. 108, January 1982, pp. 283-301.