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DEVELOPMENT LENGTH OF PRESTRESSING STRAND

A

Dissertation

**Presented for the
Doctor of Philosophy
Degree**

The University of Tennessee, Knoxville

Chong Key Chew

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Dedicated to

My Beloved Elderly Parents and All the Members of My Family,
Teachers and Friends.

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ABSTRACT

In an attempt to resolve the controversy over the adequacy of the ACI 318-83 Building Code Requirements on development length of prestressing strands, a major experimental research on bond development of seven-wire prestressing strands was conducted. Twenty-two full-scale AASHTO Type-I prestressed concrete bridge beams were tested for transfer length and strand end slip at transfer and for development length in static destructive tests. Additionally, transfer length in prestressed concrete prisms and pullout behavior of non-prestressed strands in concrete prisms were obtained.

A comprehensive review of related research is presented.

1/2" Regular, 1/2" Special, 9/16" and 6/10" strands were tested. Other variables included strand surface conditions (milled, weathered, and grit-impregnated epoxy-coated), concrete strength, strand pretension level, strand spacing, and strand manufacturers.

Transfer length was determined from strain measurements on concrete surface using a newly developed *Slope-Intercept* method. Measured transfer lengths for milled surface strands were proportional to strand diameters except the 6/10" strand which had shorter transfer length. The mean transfer lengths compared closely with the ACI code requirements. Transfer length was found to be well correlated with end slip at transfer and directly related to the strand pretension level.

Flexural bond strength was deduced from the load tests of beams that exhibited bond failure. The flexural bond strength was found to be proportional to strand diameter for all the strands. The mean flexural bond length for milled surface strands was 42% more conservative than the code requirements.

Smaller strand spacing of 1.75" for the 1/2" Regular strands did not appear to cause any adverse effects on the transfer length or the flexural bond length, indicating that the code requirement for strand spacing of 4 times the strand diameter is satisfactory. Weathered strand surface condition improved transfer length more significantly than flexural bond length. 1/2" Regular epoxy-coated strands with grit-impregnation performed considerably better than the 1/2" Regular milled strands. Concrete strength did not affect the transfer and flexural bond lengths of smaller strands but did significantly affect the larger 6/10" strands. There were nominal effects on the transfer and flexural bond lengths of the strands from different manufacturers.

The interrelation of development length with beam behavior was studied. Load capacity of beams that cracked in web-shear appeared to be governed by the available development length from the shear-crack at the face of support. Flexural-shear crack was observed to be a critical section for development length requirement evaluation.

CONTENTS

<u>Chapter</u>	<u>Page</u>
I. INTRODUCTION	1
1.1 Controversy in Code Provisions	4
1.2 Need for the Study	7
1.3 Objectives of the Study	12
II. LITERATURE REVIEW	15
2.1 Transfer Length Tests	16
2.2 Development Length Tests	23
2.3 Pullout Tests	33
III. TEST PROGRAM	37
3.1 Specimen Design and Designation	37
3.1.1 Bridge Beams	38
3.1.2 Pullout Prisms	43
3.1.3 Transfer Prisms	43
3.2 Materials	44
3.3 Fabrication of Specimens	46
3.4 Instrumentation and Test Procedure	48
3.4.1 Transfer Length Measurements	49
3.4.2 End Slip at Transfer	50
3.4.3 Pullout Tests	51
3.4.4 Development Length Tests	52
IV. TEST RESULTS	55
4.1 Bridge Beams	55
4.1.1 Transfer Length	56
4.1.2 End Slip at Transfer	61
4.1.3 Development Length Tests	63
4.2 Pullout Prisms	69
4.3 Transfer Prisms	71

V.	DISCUSSION OF TEST RESULTS	74
5.1	Effects of Variables on Bond Development	74
5.2	Transfer Length Tests	78
5.3	Development Length Tests	81
5.3.1	Interrelation of Development Length with Beam Behavior	82
5.3.2	Effects of Bond Failure on Beam Behavior	85
5.3.3	Load Capacity and Mode of Failure	90
VI.	ANALYSIS OF TEST RESULTS	95
6.1	Effects of Variables on Transfer Bond Strength	95
6.1.1	Effects of Strand Diameter	96
6.1.2	Effects of Strand Surface Condition	97
6.1.3	Effects of Concrete Strength at Transfer	98
6.2	Correlation of Transfer Length with End Slip	100
6.3	Flexural Bond from Load Tests	102
6.4	Effects of Variables on Flexural Bond Strength	108
6.4.1	Effects of Strand Diameter	108
6.4.2	Effects of Strand Surface Condition	110
6.4.3	Effects of Concrete Strength	111
6.5	Correlation of Flexural Bond with Transfer Bond	111
6.6	Effects of Variables on Pullout Bond Strength	113
6.7	Transfer, Pullout, and Flexural Bond Strengths	116
VII.	RECOMMENDATIONS	118
7.1	Comparison with Existing Prediction Equations.....	118
7.1.1	Comparison of Transfer Bond Strength Equations	121
7.1.2	Comparison of Flexural Bond Strength Equations	122
7.2	Recommended Development Length Equation	124
VIII.	CONCLUSIONS	127
	BIBLIOGRAPHY	131

APPENDICES	137
APPENDIX A THEORETICAL CALCULATIONS	138
A.1 Theoretical End Slip at Prestress Transfer	139
A.2 Relationship between the Area and the Diameter of Strand	140
A.3 Theoretical Steel Stress at Ultimate Flexural Compression Failure	141
APPENDIX B MEASURED TRANSFER STRAIN-DISTRIBUTION DIAGRAMS	148
APPENDIX C LOAD VERSUS DEFLECTION PLOTS FOR DEVELOPMENT LENGTH TESTS	177
APPENDIX D LOAD VERSUS DEFLECTION PLOTS FOR PULLOUT TESTS	200
VITA	211

LIST OF TABLES

<u>Table</u>	<u>Page</u>
2.1 - Test Specimens: Janney; 1954	17
2.2 - Transfer Length of Uncoated 1/2" Strand: Janney; 1963	18
2.3 - Transfer Length of Uncoated Strand: Kaar, LaFraugh, and Mass; 1963	20
2.4 - Transfer Length of Uncoated 1/2" Strand: Over and Au; 1965	21
2.5 - Transfer Length of Uncoated 3/8" Strand: Kaar, Hanson, Corley, and Hognestad; 1975	22
2.6 - Transfer Length of Coated & Uncoated 1/2" Strand: Dorsten, Hunt, and Preston; 1984	23
3.1 - Summary of Beam Properties	42
3.2 - Transfer Prism Specimens	44
3.3 - Properties of Prestressing Strand	45
3.4 - Concrete Mix Design	46
4.1 - Transfer Data Summary	60
4.2 - Summary of End Slip Results	62
4.3 - Development Length Test Results	64
4.4 - Pullout Test Results	71
4.5 - Transfer Prism Test Results	73
5.1 - Measured Transfer Bond Strength for Beams and Prisms	79
5.2 - Average Measured Transfer Lengths of Beams	80
5.3 - Measured Moment and Mode of Failure	91
5.4 - Strength of Beams with Bond and Shear Failures	93
5.5 - Minimum and Full Development Lengths	94

6.1 - Summary of Flexural Bond Results	103
A.1 - Summary of f_{cu}/f'_c and Steel Stress Calculations	145

LIST OF FIGURES

<u>Figure</u>	<u>Page</u>
1.1 - Development Length of Prestressing Strand	2
3.1 - AASHTO Type-I Beam Cross Section	3 8
3.2 - Shear and Confinement Reinforcement in Each End of Beam	3 9
3.3 - Strand Configurations	4 0
3.4 - Plan View of Pretensioning and Casting Bed	4 7
3.5 - Transfer Length Measurement at End of Beam	5 0
3.6 - Pullout Test Arrangement	5 2
3.7 - Loading Diagram: Development Length Tests	5 4
4.1 - Transfer Length Strain Distribution Diagram: Beams	5 6
4.2 - A Typical Flexural Compression Failure, F	6 6
4.3 - A Typical Slip and Flexural Compression Failure, B-F	6 7
4.4 - A Typical Slip and Shear Failure, B-S	6 7
4.5 - A Typical Flexural Compression and Slip Failure, F-B	6 8
4.6 - Load Versus Steel Strain for Beam Test 5-1-EXT, Strand #3	6 9
4.7 - A Typical Load Versus Deflection Relationship Pullout Test 5-1&2-36	7 0
4.8 - Transfer Length Strain Distribution Diagram: Prisms	7 2
5.1 - Interaction of Shear Crack with Development Length	8 5
5.2 - Load Versus Deflection for Beam 916-2-EXT	8 8
5.3 - Load Versus Deflection for Beam 6-2-EXT	8 8
5.4 - Load Versus Deflection for Beam 6-2-INT	8 9
6.1 - Effects of Diameter, Surface Condition, and Spacing of Strands on Transfer Bond Strength	9 7

6.2 - Effect of Concrete Strength at Transfer on Transfer Bond Strength for 6/10" Strand	99
6.3 - Effect of Concrete Strength at Transfer on Transfer Bond Strength for Milled Surface Strands	100
6.4 - Relationship between Transfer Length and End Slip	101
6.5 - Photograph of Flexural-Shear Cracks in Beam 916-1-EXT	106
6.6 - Critical Section at Flexural-Shear Crack: Beam 6-2-INT	106
6.7 - Effects of Strand Diameter on Flexural Bond Strength	109
6.8 - Effects of Strand Surface on Flexural Bond Strength	110
6.9 - Effects of Concrete Strength on Flexural Bond Strength	111
6.10 - Effects of Strand Diameter on the Correlation of Transfer Bond Strength with Flexural Bond Strength	112
6.11 - Effects of Strand Diameter on Pullout Bond Strength	114
6.12 - Effects of Concrete Strength on Pullout Bond Strength of 6/10" Strand	115
6.13 - Effects of Concrete Strength on Pullout Bond Strength	116
7.1 - Comparison of Measured Transfer Bond with Existing Equations	121
7.2 - Comparison of Measured Flexural Bond Strength with Existing Equations	123
A.1 - Strain Distribution in Prestressed Concrete Beam	139
A.2 - Relationship between the Area and the Diameter of Strand	140
A.3 - Conditions of Strain and Stress for Beam Section	142
A.4 - f_{cu}/f_c Versus k_d for All Beams with Flexural Failure	146

LIST OF SYMBOLS

<u>Symbol</u>	<u>Description</u>
A_{ps}	Area of prestressed reinforcement (in. ²)
B	Bond modulus (psi/in.)
C	Compressive force in the concrete (kips)
C_b	Distance from bottom of member to neutral axis (in.)
d_b	Nominal diameter of prestressing strand (in.)
E_s	Modulus of elasticity of steel (psi)
F_{pu}	Specified ultimate strength of prestressing strand (ksi)
f'_c	Compressive strength of concrete (psi)
f'_{ci}	Compressive strength of concrete at time of initial prestress (psi)
f_{ps}	Steel stress at nominal beam strength (ksi)
f_{sb}	Stress in the strand at general bond slip (ksi)
f_{se}	Effective stress in prestressing steel after losses (ksi)
f_{si}	Initial prestress in steel reinforcement (ksi)
f_{sm}	Maximum steel stress measured at test (ksi)
I	Moment of inertia of a beam section (in. ⁴)
L_b	Flexural bond length of strand (in.)
L_d	Development length of strand (in.)
L_e	Distance from end of beam to point of maximum moment in a beam (in.)
L_t	Transfer bond length of strand (in.)
M_b	Moment at bond failure (k-ft)
M_{cr}	Cracking moment of a section (k-ft)

M_u	Measured moment strength of a section (k-ft)
M_n	ACI nominal moment strength of a section (k-ft)
P	Applied transverse load on a beam (kips)
R	Support reaction force on a beam (kips)
T	Tensile force in the steel of a beam (kips)
U_a	Average bond stress over embedment length (ksi)
U'_d	Average bond stress over the flexural bond length (psi)
U'_t	Plastic transfer bond stress (psi)
Δ	measured initial strand slip (in.)
Δ_{ct}	proposed limiting measured amount of slip (in.)
Σ_p	Total circumferential area per inch of strand length (in. ²)

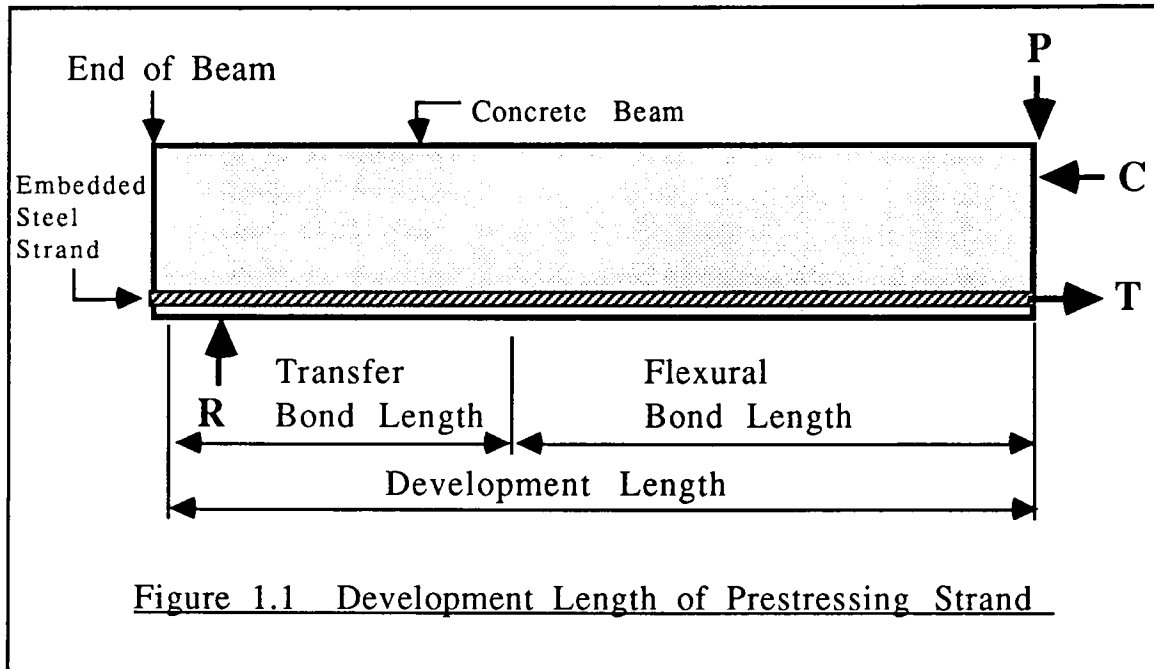
Chapter I

INTRODUCTION

Twisted seven-wire high-strength steel strand is the dominantly used tensile reinforcement in the prestressed concrete construction industry. In unbonded post-tensioned prestressed concrete members, only minimal shear resistance exists between prestressing steel and concrete along the length of the steel, and the transfer of the tensile forces of the steel to the concrete is accomplished with the use of costly mechanical end anchorages at both ends of each member. In contrast, steel strands in *pretensioned* prestressed concrete members are cast compositely with the concrete and rely solely on the bond resistance of the concrete surrounding the strands to develop the necessary tensile stresses in the strands. The research herein focuses on the bond development of seven-wire 270 ksi high-strength low-relaxation prestressing steel strand in pretensioned prestressed concrete products, particularly the bond development in flexural members.

The length over which the bond resistance exists to build the steel stress at a section of a member is called the "development length" of the strand. As illustrated in Figure 1.1, the steel stress that a strand can achieve, to develop a flexural moment at a beam section, is dependent upon the available development length of the strand from the beam section. Full development length is defined

herein as the development length necessary for developing the steel stress needed to achieve the ultimate flexural strength of a beam section.



The bond resistance in a prestressed concrete beam arises from two distinct situations: from the anchorage of prestressing force, and from the change in steel force along the length of the steel, due to moment gradients created by transverse loadings.

Pretensioned prestressed concrete beams are normally pre-fabricated in a prestressed concrete plant and shipped to a construction site for use. The fabrication process involves initial prestressing of steel strands, then casting of concrete, and finally detensioning of the strands. Before the concrete is cast around the strands in a form, the strands are stretched to a predetermined tension against two fixed abutments of a casting bed. The initial

tensile stress in each strand causes the strand to shrink diametrically due to Poisson's ratio. Internal vibration of the surrounding concrete causes it to set into the grooves between the wires of the strands and to adhere to the surfaces of the wires.

When the concrete has been cured to a certain minimum compressive strength, the strands are released from the abutments. At both ends of the beam, the stress in each strand immediately drops to zero, resulting instantaneously in a sharp stress gradient in the steel. The strand simultaneously withdraws and expands diametrically, wedging against the restraining concrete and increasing its resistance to sliding. When the ultimate adhesive bond resistance is overcome and replaced simultaneously by the frictional and mechanical bonds, the strand slips into the concrete and comes to equilibrium under a sustainable stress gradient. Progressive slippage like this occurs over some length from the end of the beam. While the bond resistance around each strand builds up enough force over a distance to stabilize the strand, the total force from all the strands compresses the concrete and reduces the stress in the strands altogether. Some of the initial steel stress is lost as a result of the elastic shortening of the concrete. The bond resistance at the ends of the beam is specifically called the "transfer bond", and the distance from each end of the beam over which the bond exists is referred to as the "transfer length."

When the pretensioned prestressed beam is placed into service and transversely loaded, additional bond called "flexural bond" is

induced along the strands as a result of the difference in stiffness of steel and concrete and the moment gradient along the beam. The flexural bond is relatively small before any cracking of the beam. However, at the first flexural crack, a high bond resembling a "pullout bond" in nature comes into existence immediately in the vicinity of the crack. As transverse load increases, the pullout bond progresses away from the crack and toward the end of the beam. Additional flexural cracks may form closer to the end of the beam. With each additional flexural crack, the pullout bond that existed between the cracks is transmuted to the flexural bond of cracked sections. The flexural bond of cracked sections is higher in magnitude than that of uncracked sections, but it may be lower than the pullout bond. The pullout bond then progresses further toward the end of the beam, and may enter into the transfer bond zone and superimposes with the transfer bond. When the ultimate bond strength over any length of the strand from the end of beam is exceeded, a "bond failure" occurs, indicated by end slippage of the strand. It has been observed that higher bond strength is attainable after the end slippage has occurred, but such strand slippage often precipitates a premature flexural compression failure at the critical section or shear failure in the span at a considerably lower load.

1.1 Controversy in Code Provisions

The earliest major tests of development length of high-strength seven-wire prestressing steel strands were conducted by Hanson and

Kaar in 1959 [15]. The test program involved a total of 47 test beam specimens and included three strand sizes of 1/4", 3/8", and 1/2" in diameter. Based primarily on these experimental results, provisions for development length were first incorporated into the American Concrete Institute (ACI) 318-63 building code and subsequently adopted by the American Association of State Highway and Transportation Officials (AASHTO) bridge design code [34]. The provisions have remained unchanged up to the most recent editions of these codes.

Few other similar experimental research projects have been performed since 1959, and not all of the results support the current code provisions. Anderson and Anderson [1] in 1976, and Brooks, Gerstle and Logan [3] in 1988, demonstrated separately with tests of 8" hollow-core slabs that if excessive end slips of the strands occurred at transfer, the code provisions overestimated the ultimate strength of the slabs. The slabs were prestressed with strands up to 1/2" in diameter. Different approaches for adjustments to the provisions were proposed to account for the excess strand end slips.

The experimental results most contradictory to the code provisions were reported in December of 1986 by Cousins, Johnston, and Zia [6,7,8] of North Carolina State University. The results obtained in this study indicated that the development length predicted by the code provisions was severely unconservative, especially for larger strand sizes. Although the experiment on uncoated strands was limited in extent, it has raised doubts about

the adequacy of the code provisions. Epoxy-coated strands were studied in greater detail and shown to have better bonding properties than the uncoated strands. Analytical expressions based on an assumed elasto-plastic bond stress distribution were developed from the experimental data of these and other tests in the literature for estimating the transfer length and the development length of the uncoated and coated strands.

In 1988, another test program by Maruyama and Rizalla [26] with pretensioned thin-web T-beams having low shear-span to beam-depth ratio and high beam shear stresses also found the code prediction equation for development length inadequate. The steel stress developed by the strands appeared to correspond to that calculated using the ACI equation for the development length from the diagonal cracks crossing the strands. Others like Martin and Scott [25] in 1976 and Zia and Mostafa [39] in 1977 independently proposed two other equations, considerably more conservative than the ACI equation, for the development length after the reevaluations of applicable past experimental data.

The apparent contradiction in the research data has shaken the the industry's confidence in the code provisions. As a result, the Federal Highway Administration (FHWA) has responded with restrictions on the use of prestressed concrete products. FWHA is particularly concerned with the safe use of strands larger than 1/2" in diameter and has banned the use of 6/10" diameter strand. FHWA has also imposed stringent strand spacing requirements. As

an interim measure until further research indicates otherwise, FHWA has specified that the design development length is to be 2.5 times that given by the current AASHTO equation. A joint committee of FHWA, AASHTO, and the Prestressed Concrete Institute (PCI) was later formed to discuss the problem, and the joint committee decided that the design development length should be increased to 1.6 times that of the AASHTO equation. These added restrictions and requirements have had a severe negative impact on the competitiveness of the prestressed concrete industry.

1.2 Need for the Study

A new test program was needed to resolve the conflict and restore confidence in the code provisions. With the introduction of epoxy coated strands for corrosion protection, and the desire to use larger diameter strands for greater economy, more studies and representative experimental data were clearly needed. At the request of PCI, a research team at the University of Tennessee at Knoxville (UTK) tailored a test program specifically to meet this need.

No previously obtained experimental data have supported extending the use of the code prediction equation for strands larger than 1/2" in diameter. The UTK test program includes the 1/2" strand and three larger strand diameters of 1/2" special, 9/16", and the 0.6" banned for use by FHWA. These experimental results serve not only to test the validity of any of the prediction equations for the

1/2" strand previously studied, but also to define the extent of applicability for the strands of larger diameters.

Many of the previous experiments on development length, including the research by Hanson and Kaar [15], used strands that were cleaned of residual surface oil. These results may not be representative of the actual bond strength of the new strands received from steel mills. A milled (as received, shiny) surface condition provides a less favorable bonding surface and thus a lower-bound bond strength. On the other hand, weathered strand surfaces have been consistently observed to enhance bond strength significantly. The epoxy-coated strands, with coating surfaces impregnated with crushed glass, were observed by Cousins, et. al. [6,7,8] also to have higher bond strength. The UTK test results for all of these strands are used to test past data and recommendations for the similar strands.

Although the concrete is intimately associated with the steel strand in a beam and provides the bond resistance for the strand, the effect of concrete strength on the bond strength is not fully understood. Kaar, LaFraugh, and Mass [20], in an extensive investigation of the effect of concrete strength on transfer length, concluded that there is no systematic variation in transfer length with concrete strength. However, some other investigators [14,23,32,35] have found some proportionality of increased bond strength with increased concrete strength. Cousins, et. al [6] have simply assumed that the bond strength is proportional to the square-

root of the concrete strength. The UTK test program, although not designed to investigate the effect of concrete strength, has specimens varying in concrete strength within a range likely to be encountered in practice; these specimens are examined for any effect of concrete strength.

The effects of different strand pretensions on transfer bond are investigated so that a direct comparison can be made to the results obtained by other investigators for strands with the same level of pretension. All beams in the UTK test program had an initial pretension of 202-ksi, which is higher than all other test programs except that by Cousins et. al [6]. It was suspected that the longer development length obtained by Cousins et. al. [6] may be due to the higher pretension used in their experiment.

Strand spacing requirements are important to the design of prestressed concrete members. Strands placed too close together can cause splitting of the surrounding concrete between the strands, resulting in a great loss of bond. No previous study has established a criterion for minimum strand spacing. FHWA, in the midst of the controversy, has temporarily imposed a strand spacing requirement of 4 times the strand diameter. In the present test program, a limited study of the effects of reducing the strand spacing from 2" to 1.75" for the 1/2" strands provides some basis for establishing such a criterion. Also explored are the effects on bond strength of possible mechanical and physical variations in the 1/2" diameter strands produced by four different manufacturers.

Most prestressed concrete flexural members are thin-web shaped and thus very susceptible to flexural-shear and/or web-shear crackings, especially for a short span member where development length is most likely to be critical and shear force most intense. In general design practice, shear and flexural strengths are treated separately. Attention is commonly focused on the availability of development length of the strand to develop the steel stress necessary to achieve a certain flexural strength at a critical section, and shear reinforcements are designed to resist the corresponding shear force when the beam web cracks in shear at overload. Not enough consideration has been given in the literature to the effects of the shear cracking on the strand development length, which consequently affects the beam flexural strength. Shear cracking transforms a concrete beam into a "truss", a new structural configuration with a different stress transfer mechanism. The truss analogy of a beam cracked in flexural-shear and/or web-shear dictates a more critical development length requirement than a beam without the cracks. Prestressing is able to delay cracking, but it also increases the load at cracking. For a beam with flexural strength governed by the available development length, the cracking may transform the beam into one that has a lower load capacity governed by the reduced development length after the cracking. Neglecting the effects of the cracking on development length may lead to a design without adequate strength and ductility.

Several different modes of failure have been observed of the test beams in the UTK test program. The load versus deflection behavior of some I-beams in the UTK test program that suffered web-shear cracking, most probably triggered by some strand slippage, reveals a very interesting phenomenon. These beams lost load capacity abruptly when web-shear cracks began to form, and the maximum resistance of each beam dropped to a significantly lower sustainable load. It is also interesting to note that this lower load corresponds approximately to the load calculated using the truss analogy for the development length of the strands from the web-shear crack nearest to the end of the beam. This phenomenon has significant design implications, but it has not been investigated in the past. The behavior of these beams is explored for the necessary requirements regarding development length (which may be called "anchorage length") of strands from the shear crack to the end of beam.

Flexural stiffness of beam reduces greatly with flexural cracks, and the danger of bond failure becomes more eminent for beams without sufficient development length. Under intense shear force over the shear span, flexural cracks in a beam with bond failure may develop into a flexural-shear crack, which reduces the effectiveness in resistance of a bond length within the flexural bond length. This ineffective bond length reduces the available development length for a critical beam section. Therefore, for beams with high design shear that may result in flexural-shear cracks, which is a typical situation

for short thin-web beams, development length requirements may need to be more stringent. The UTK beam tests revealed that the critical section for these beams is at the flexural-shear cracks, instead of the assumed section at the point of maximum moment. Thus, the available development length of the bottom strands may be shortened by the distance between the flexural-shear crack and the point of maximum moment.

1.3 Objectives of the Study

Because of the controversy over the adequacy of the existing ACI code equation [4], and the many other proposed equations for prediction of development length of prestressing strands, the reliability of the development length prediction equations evidently needs to be reexamined. A careful consideration of the problem reveals several specific details that need reevaluation. All of the equations are based very simply on a bi-linear bond distribution consisting of the draw-in ultimate transfer bond and draw-out ultimate flexural bond portions. However, the bond strengths implied by the equations vary considerably. The variables included to modify the bond strength also differ among the equations. Through the present research, on the basis of the experimental results, it is hoped that the current ACI code equation can be further substantiated or revised. If the data indicate that revisions are appropriate, alterations to the code provisions will be proposed.

Several variables in the UTK test program that may have influence on the development length are identified and outlined below:

- a) Strand Diameter (1/2", 1/2" special, 9/16", 0.6")
- b) Strand Surface Condition (milled, weathered, epoxy-coated)
- c) Concrete Strength
- d) Strand Pretension (175ksi, 202.5ksi)
- e) Strand Spacing (2", 1.75")
- f) Strand Manufacturer (four different ones)

These variables, some of which have not before been investigated, are individually evaluated in detail to determine their effects on the bond strength along the development length. Recommendations are made as to which of these variables may be routinely included to improve development length predictions. Furthermore, the interrelation of development length with beam behavior is examined in detail. The amount of end slip measured at transfer is studied for its relationship to the transfer length and for the possibility of its use as a quality control measure as proposed by some previous investigators [1,3]. Flexural bond strength is also evaluated for any correlation with transfer bond strength.

The objectives of the present research are: 1) to identify factors affecting bond development in a development length and evaluate its effects, 2) to contribute to the resolution of the controversy over the adequacy of the existing ACI code equation and the several other proposed equations for prediction of development length of seven-

wire prestressing strands, and 3) to ultimately recommend a substantiated or revised prediction equation that the prestressed concrete industry can use conveniently and confidently.

Chapter II

LITERATURE REVIEW

Since 1950, considerable research on the bond between seven-wire prestressing steel strand and concrete in prestressed concrete products has been reported. Most of the research was on the bare strands. Due to the need for corrosion protection of the bare strands for certain structures, like bridge members exposed to de-icers and salt, epoxy-coated strands were developed by Florida Wire and Cable. Two experimental studies on epoxy-coated strands have been reported since the mid-1980's.

No analytical model has yet been available that can satisfactorily predict the bond development of strands in a development length. Various test arrangements have been used to study experimentally the bond behavior in the composite structures of the two materials. These tests can be generally separated into three functional test categories: 1) transfer length tests, 2) development length tests which include tests for flexural bond, and 3) pullout tests. Many variables have been observed to affect the bond strength, but not all of them are in good agreement. This review summarizes the theories, prediction equations, test programs, test results, and conclusions by each investigator.

2.1 Transfer Length Tests

In 1954, J.R. Janney [18] conceptualized the bond between prestressing steel and the surrounding concrete to consist of three factors: 1) adhesion, 2) friction, and 3) mechanical resistance. Because of a finite amount of slip at the ends of beams at prestress transfer and the fact that adhesion between concrete and steel could exist only before steel slips, Janney reasoned that friction and mechanical interlock must be responsible for the transfer of steel forces to the concrete. Due to normal forces created by the swelling of steel in the end regions of a beam at prestress transfer, Janney further deduced that friction was probably most responsible for transfer of prestressing force. An elastic theoretical analysis for steel in thick-wall concrete cylinder was presented, which indicated that the concrete radial tensile stress, due to the normal forces from the swelling of steel, was well beyond the elastic tensile range of the concrete. Having recognized that the calculated friction stress distributions from the elastic theoretical analysis were invalid, Janney compared them to the observed stress distributions from transfer prism tests, and found them to be similar in distribution but not magnitude; the similarity led him to believe that frictional bond was mainly responsible for the force transfer.

Janney investigated two types of specimens for the bond characteristics along the length of prestressed steel: transfer prisms and short flexural beams. The effects on bond of several variables

were investigated, including wire sizes, surface conditions, pretensioned force, and concrete strength. Table 2.1 summarizes the steel sizes and surface conditions tested. Each transfer prism was 2" x 2" in cross section and between 72" and 96" long, and contained a single prestressing wire or strand. The beam specimens were 6" x 10" in cross-section. The distribution of prestress was measured on the concrete surface. Janney observed that rusted wire could develop the full prestress transfer at a more rapid rate and in a shorter distance. Concrete strengths from 4500 psi to 6500 psi, and wire diameters from 0.100" to 0.276" were observed to affect transfer length slightly. Strain gages installed at intermediate points along the length of the wires revealed the progression of flexural bond stress toward the ends of a beam, threatening a bond failure when the flexural bond stress reaches the transfer zone.

Table 2.1 - Test Specimens: Janney; 1954			
Wire	Surface Condition		
Diameter (in)	Clean	Lubricated	Rusted
0.1	X	X	
0.162	X	X	X
0.197	X	X	
0.276	X	X	
5/16" Strand	X	X	

In 1963, Janney [19] also conducted tests on transfer characteristics of 270 ksi seven-wire prestressing strands of 1/2" in diameter. Six concrete specimens of 3.5" x 4.25" x 8'-0" were each concentrically prestressed with a strand: two specimens with 250 ksi clean and shiny strands, two with 270 ksi clean and shiny strands, and two with 270 ksi strands that had a medium coat of rust. Table 2.2 summarizes the transfer length results for the three conditions. The author concluded that the transfer length is slightly greater for high strength strand at the higher initial pretension force. As a result of the satisfactory performance of the 1/2" strand, Janney presumed that smaller diameter 270 ksi strand would perform satisfactorily in prestress transfer for all normal applications.

Table 2.2 - Transfer Length of Uncoated 1/2" Strand: Janney; 1963				
Ultimate Strength (ksi)	Surface Condition	f _{se} (ksi)	f' _c (psi)	Transfer Length Measured (in)
270	Clean	175.8	4115	33
270	Rusted	175.8	4090	24
250	Clean	150.0	4200	28

In 1963, Kaar, LaFraugh, and Mass [20] studied the effects of concrete strength at transfer in an attempt to determine the minimum allowable concrete strength for transfer. The test program included four strand diameters: 1/4", 3/8", 1/2", and 6/10", and five concrete strengths at transfer: 1660, 2500, 3330, 4170, and 5000 psi.

All strands were clean, except the 6/10" strands which were slightly rusted before cleaning. The ultimate strengths of the strands ranged from 253 ksi to 275 ksi, and the effective prestress immediately after transfer varied from 0.58 to 0.72 of the ultimate strengths. The specimens were rectangular concrete cross sections, having a certain cross-section for each strand diameter ranging from 3" x 4-3/16" for 1/4" to 7-1/2" x 10-1/2" for 6/10". The number of strands in a specimen varied from two to six strands with concrete strength at transfer. Transfer length was measured with a Whittemore gage and brass discs. Transfer readings were taken on 1, 3, 7, 14, 28, 56, 90, 180, and 365 days after transfer. Table 2.3 is a partial summary of the results for 3/8", 1/2", and 6/10" diameter strands with concrete strengths at transfer of 4170 and 5000 psi. All transfer lengths reported were adjusted to reflect an effective prestress of 175 ksi (or $0.7f_{pu}$). The authors concluded that there was no systematic variation in transfer length with concrete strength. For strands up to 1/2" in diameter, concrete strength had little influence on transfer length. For 6/10" diameter strand, transfer length decreased with an increase in concrete strength at the cut end, but not so obviously at the dead end. Transfer length was observed to be almost proportional to strand diameter. Transfer length at cut ends was higher than at dead ends by approximately 20% for 1/2" diameter strand, and 30% for 6/10" diameter strand. The average increase in transfer length over one year was 6%. In three auxiliary tests with 3/8" strands having steel stresses after transfer of 120 ksi to 175

ksi, transfer length was found to be proportional to the steel stress. Smooth wire spiral placed concentrically about each strand was observed to decrease transfer length about 18% at the cut end, and 9% at the dead end.

Table 2.3 - Transfer Length of Uncoated Strand: Kaar, LaFraugh, and Mass; 1963					
Strand	Concrete	Type	Transfer Length (in)		
Diameter	Strength	of	1	28	365
(in)	(psi)	Release	Day	Days	Days
3/8	4170	Sudden	---	---	---
		Gradual	---	---	---
	5000	Sudden	25.5	26	27.5
		Gradual	20	23	25.5
1/2	4170	Sudden	37	35.5	37.5
		Gradual	34	34	38
	5000	Sudden	41	42	44
		Gradual	34	36.5	37.5
6/10	4170	Sudden	39	38.5	40.5
		Gradual	31.5	32.5	33.5
	5000	Sudden	39.5	40.5	39
		Gradual	28.5	30	32

In 1966, Over and Au [27] reported their transfer length test results for three strand diameters of 1/4", 3/8" and 1/2", having an effective prestress from 164 ksi to 170 ksi. The results are tabulated in Table 2.4. From these results, the authors concluded that transfer length is greater for larger diameter strands.

In 1969, Hanson [16] studied the effect on transfer length of strand surface roughness: clean, partially rusted, rusted, and experimentally deformed strand. Improvement in transfer length and ultimate flexural strength was observed for rusted and deformed strands.

Table 2.4 - Transfer Length of Uncoated 1/2" Diameter Strand: Over and Au (1966)				
Strand Diameter (in.)	Size of Prism (in.)	Concrete Strength (psi)	fse (ksi)	Transfer Length (in.)
1 / 2	3x3x80	5500	170	35
3 / 8	3x3x60	4180	160	30
1 / 4	3x3x60	4900	164	20

In 1975, Hanson, Kaar, Corley, and Hognestad [17] studied the effects of three surface conditions of 3/8", 270 ksi prestressing strand: sandblasted, lightly rusted, and smooth. Initial prestress of between 180 ksi and 185 ksi was transferred at a concrete compressive strength of 4000 psi with either sudden or gradual releases. Changes in concrete strain were measured with a Whittemore gage and brass discs. Table 2.5 summarizes the acquired transfer lengths, which were defined as the distances to build up 95% of concrete strain plateau. The results were widely scattered for each surface condition and type of release. On the average, the transfer length of the smooth strand with sudden release was about

20% longer than that with gradual release, and the slightly rusted and the sandblasted strands respectively had transfer lengths about 51% and 36% shorter than the smooth strand.

Table 2.5 - Transfer Length of Uncoated 3/8" Strand: Kaar, Hanson, Corley, and Hognestad; 1975					
Surface Condition	Type of Release	Number of Tests	Transfer Length (in)		
			Low	Avg.	High
Smooth	Gradual	39	12	23.9	39
Smooth	Sudden	37	17	29.2	54
Slightly Rusted	Sudden	12	9	14.2	23
Sand Blasted	Sudden	15	11	18.6	28

In 1984, Dorsten, Hunt, and Preston [11] reported their test results on transfer lengths for seven pretensioned concrete prism specimens: three with grit-impregnated epoxy-coated strands, and four with bare strands. The grit density of the coated strands was designed to give a bond comparable to that of uncoated strands. The specimens were 3-1/2" x 4-1/2" in cross-section and 8' long. All the strands were 1/2" diameter low relaxation 270 ksi strands. The initial prestress of $0.75f_u$ was transferred at a concrete strength of 4000 psi. Table 2.6 summarizes the test results. The cut and dead ends respectively referred to the end where the strand was cut and the other end of a specimen. Just after transfer, the average transfer length for uncoated strands was 26.9", and that for coated strands was 31.3", an increase of 16%. After 14 months the transfer length for coated strand was 3% less than that of the uncoated strand.

Splitting that occurred at transfer in some epoxy-coated specimens led the authors to conclude that more cover might be required for epoxy-coated strands.

Table 2.6 - Transfer Length of Coated and Uncoated 1/2" Strand: Dorsten, Hunt, and Preston; 1984				
Strand Type	End of Specimen	Number of Tests	Transfer Length (in)	
			Initial	14 Months
Bare	Dead	4	26.3	33
	Cut	4	27.5	33.5
Coated	Dead	3	33.3	35
	Cut	3	29.3	29.7

2.2 Development Length Tests

In 1959, Hanson and Kaar [15] tested 47 rectangular beams that were prestressed with 1/4", 3/8", or 1/2" seven-wire prestressing strands having an ultimate strength of 250 ksi. The beams' "width" x "effective depth" ranged from 4" x 5.56" to 8" x 11.47". All strands were cleaned of surface oil by washing in carbon tetrachloride, except rusted strands which were weathered for a week in a moist environment. Prestressing force was gradually released at a concrete compressive strength of 4500 psi, except for a few lower concrete strength specimens which had 3500 psi. Effective prestress in the strands was between 113 ksi and 148 ksi. All specimens were loaded either at midspan or two points located symmetrically about midspan. Strain gages at intermediate points in the length of beam measured changes in steel stress during loading.

Dial gages measured deflection of the beam and strand slip at the ends of the beam.

The measured change in steel stress along the length of strand by strain gages was described. Prior to cracking, the strand stress increased only slightly. After cracking, the strand stress increased dramatically in the vicinity of the crack. With a further increase in the load, the stress increase moved as a wave toward the transfer region. When the stress wave reached the transfer region, a general bond slip occurred, and the strand stress decreased. Despite general bond slip, mechanical interlock and friction in the development length were enough to maintain a substantial stress in the strand to enable the load to increase further, until a flexural failure occurred.

Two modes of failure were observed. Thirteen beams failed in flexure with no strand slippage. Thirty-four beams failed in flexure after general bond slip of the strand was observed. From these test results, the minimum required embedment lengths to fully develop the ultimate moment without any end slips were found to be 70", 106", and 134" for 1/4", 3/8" and 1/2" diameter strands respectively. Because the steel stress at flexural failure is lower for a beam with higher percentage of steel or lower concrete strength, the authors concluded that the chance for a bond failure is lower for a beam with higher percentage of steel or lower concrete strength. Rusted strand improves moment at general slip ranging from insignificant to as much as 53%, with an average of 30%. Specimens with end anchors did not improve moment at general slip.

Based on the test results by Hanson and Kaar, an empirical relationship was adopted into the ACI code in 1963 and the AASHTO code in 1983. The equation appears in section 12.9.1 of the ACI 318-83 code [4], and the AASHTO code [34] section 9.27.1 in the following form:

$$L_d = (f_{ps} - 2/3 f_{se})d_b \dots\dots\dots (2.1)$$

where:

- L_d = development length (in),
- f_{ps} = stress in prestressed reinforcement at nominal strength (ksi),
- f_{se} = effective stress in prestressing steel after losses (ksi),
- d_b = nominal diameter of prestressed reinforcement (in).

In the ACI 318R-83 code commentary, equation (2.1) was rewritten into the following form:

$$L_d = (f_{se}/3)d_b + (f_{ps} - f_{se})d_b \dots\dots\dots (2.2)$$

In equation (2.2) the first term, $(f_{se}/3)d_b$, is the transfer length that develops f_{se} , and the second term, $(f_{ps} - f_{se})d_b$, is the additional bond length necessary to develop an increase in steel stress of $(f_{ps} - f_{se})$.

In 1965, Kaar and Magura [21] tested five 34-ft half-scale Type III AASHTO-PCI girders to study the effects of strand blanketing on flexural behavior and shear capacity. Twelve seven-wire 3/8" strands in each girder had an initial prestress of 175 ksi and a calculated effective prestress of 140 ksi. Blanketing to prevent bond in the end regions of the beam was achieved with polyethylene tubing covering around the strands. Loading was applied to the

girders at four points symmetrical about the mid-span. Three girders, designed for flexural behavior study, were overly-reinforced for shear and subjected to 5 million cycles of service load prior to a static test to failure; the first one with fully bonded strands was a control girder, the second one with "partial blanketing" had just twice the development length computed using equation (2.1) for bonded strands, and the third one with "full blanketing" had just the development length computed using equation (2.1) for bonded strands. The other two girders, designed for shear behavior study, had stirrups spaced at 1-1/2 times that required by the ACI code; one girder had strands fully bonded, and the other had "partial blanketing."

For the three flexural girders, no crack or bond failure was observed after the 5 million cycles of service load. The fully blanketed third girder failed by strand fracture at an outer load point at 84% of calculated ACI nominal flexural strength at the section; the first and second girders had 98% and 96% of calculated nominal strength, respectively. Flexural cracks for the third girder were much wider than for the first and second, indicating a bond deficiency in the third girder. Strain gage readings confirmed a bond failure in the third girder. In the two shear girders, the girder with fully bonded strands failed in horizontal shear at 88% of calculated ACI nominal shear strength; the other failed in strand fracture near mid-span at 101% of calculated ACI nominal flexural strength, and the shear force at the failure was 84% of calculated nominal shear

strength. All blanketed strands in the shear girder suffered bond failure. The authors concluded that 1) blanketing had no detrimental effects on shear strength, 2) the ACI development length requirement was not adequate for blanketed strands, and 3) performance of girder with twice the development length for blanketed strands was comparable to that with fully bonded strands.

In 1976, A. R. Anderson and R. G. Anderson [1] evaluated the flexural bond performance of 36 pretensioned hollow-core slabs. The test specimens were produced by five different manufacturers. Each slab was 8" in depth with 1/2" diameter 270 ksi strand pretensioned to $.7f_{pu}$. The concrete strength was approximately 5000 psi and the release was sudden by saw-cutting. The authors derived the following relationship between flexural bond performance and free end slip:

$$\Delta_{ct} = f_{si}d_b/950 \dots\dots\dots (2.3)$$

where:

Δ_{ct} = proposed limiting measured amount of slip (in)

f_{si} = initial jacking stress (ksi)

The authors concluded that flexural bond performance is directly related to the amount of free end slip. In twelve of fifteen tests that failed in flexural bond, the first strand to slip was the one with the largest amount of free end slip. Twenty four specimens met or exceeded the ACI code when the measured free end slip was less than that calculated in their prediction equation. Poor consolidation

provision. It was also concluded that the double development length requirement for debonded strand in the code section 12.9.3 was also based on reliable experimental evidence, noting that although less than double may be adequate, there are no test results to support the fact and thus to justify any possible relaxation of the provision.

In 1986, Cousins, Johnston, and Zia [6,7,8] researched the transfer and development lengths of 60 beams prestressed with uncoated, smooth epoxy-coated, and glass-impregnated epoxy-coated strands of 3/8", 1/2" and 6/10" in diameter. Transfer length was determined by measuring the strain on the concrete surface before and after release, while the development length was determined by a series of static tests. From their results, the authors recommended the following equations:

For transfer length, L_t :

$$L_t = (.5U'_t\sqrt{f'_c})/B + f_{se}A_s/(\pi d_b U'_t\sqrt{f'_c}) \dots\dots\dots (2.8)$$

For flexural bond length, L_b :

$$L_b = (f_{ps} - f_{se})(A_s/\pi d_b)/(U'_d\sqrt{f'_c}) \dots\dots\dots (2.9)$$

where:

U'_t = plastic transfer bond stress (psi)

f'_c = compressive strength of concrete (psi)

B = bond modulus (psi/in)

A_s = area of prestressing strand (in)

U'_d = average bond stress over flexural bond length (psi)

The U'_t and U'_d determined from their experimental results for strands of all diameters were 6.7 and 1.32 for uncoated strands, 16.5

and 6.4 for medium to high grit density coated strands, and 10.6 and 4.55 for low grit density coated strands. The B value was highly variable, but an average value of 300 psi/in. was recommended. The authors concluded that 1) chuck slip for coated strand is considerably greater than that for uncoated strand, and may continue to slip up to 24 hours after tensioning; 2) the glass-impregnated epoxy strand had shorter transfer and development lengths than required by the ACI and AASHTO, the uncoated strands did not meet the code, and epoxy coated strand without glass has virtually no bond to the concrete; 3) measured end slip of the strands could be used to confirm transfer lengths measured from concrete surface; 4) transfer length with a gradual release is 6% lower than that with a sudden release, and 5) longitudinal splitting which occurred in two specimens with epoxy-coated strand at release indicates possible splitting problems in members with close spacing, thin cover, thin element, or poor confinement.

In 1988, Brook, Gerstle, and Logan [3] combined Anderson and Anderson's theory of bilinear transfer stresses with Robert F. Mast's theory of bilinear strand development diagrams to compose the concepts of what the authors referred to as the "Slip Theory". This theory directly relates bond quality of the concrete to the initial end slip of the strand. Their tests involved 11 hollow-core prestressed slabs 8" deep, 48" wide, with varying lengths up to 27'-0". The prestressing reinforcement was a combination of 3/8" and 1/2" diameter 270 ksi strand, initially pretensioned to $0.6f_u$ and $0.7f_u$

of the concrete around the strands was identified to be the primary cause of excessive free end slip and premature flexural bond failure.

In 1977, Salmon and McCrate [31] studied the bond behavior of untensioned prestressing strand on 69 beam specimens that consisted of straight, frayed, and 90° bent strand configurations, and different concrete strengths and strand diameters. Design equations were developed for each of the three strand configurations. Concrete strengths from 3750 to 6900 psi were found to have little influence on bond at general slip. Strand diameters of 3/8", 7/16", 1/2" and 6/10" were considered to have no apparent effects on steel stress at general slip.

In 1977, Zia and Mostafa [39] developed an empirical transfer length based on transfer length research before 1977, and a flexural bond length equation based on a reevaluation of Hanson and Kaar's test results from 1959:

For transfer length, L_t

$$L_t = [1.5(f_{si}/f_{ci})d_b] - 4.6 \dots\dots\dots (2.4)$$

and flexural bond length, L_b :

$$L_b = 1.25(f_{su} - f_{se})d_b \dots\dots\dots (2.5)$$

where:

f_{ci} = concrete strength at transfer (ksi)

f_{su} = ultimate stress of the prestressing strand (ksi)

The equation for the transfer length differs from other equations because it allows for adjustment for different concrete strengths at

release, different strand sizes, and different initial prestressing. This equation is comparable to the ACI code equation for small strands, but is more conservative for the larger strands. Zia and Mostafa concluded that the flexural bond length specified in the ACI code needed to be increased 25%.

In 1979, Martin and Scott [25] proposed a revision to the code equation for design of members too short to provide the full development length. A bilinear curve fitting the data by Hanson and Kaar's research of 1959 was proposed:

For $L_x \leq 80d_b$:

$$f_{ps} < L_x/80d_b[(135/d_b^{1/6}) + 31] \dots\dots\dots (2.6)$$

For $L_x \geq 80d_b$:

$$f_{ps} < (135/d_b^{1/6}) + (.39L_x/d_b) \dots\dots\dots (2.7)$$

where:

L_x = distance from end of beam to point of consideration (in)

In no case shall f_{ps} be greater than that of equation 18-3 of the ACI 318-83 code. The authors recognized that the results based on the gradual release used in Hanson and Kaar's research was unconservative for most situations which used flame-cut or saw-cut release.

In 1986, Ghosh and Fintel [13], on a review of existing information, concluded that the ACI 318-83 code provision on "Development of Prestressing Strands" was based on good experimental authority, and recommended no change to the

respectively. Strand release was by saw-cutting when the concrete strength reached 4000 psi. From the equations (2.2), the authors derived the following equations:

For transfer length, L'_t :

$$L'_t = 2\Delta E / f_{si} \dots\dots\dots (2.10)$$

For flexural bond length, L'_b :

$$L'_b = (f_{ps} - f_{se})L'_t / f_{se} \dots\dots\dots (2.11)$$

where:

Δ = measured initial strand slip at ends (in)

E = modulus of elasticity of the strand (psi)

In 1988, Maruyama and Rizkalla [26] tested nine pretensioned and two non-pretensioned tee beams with identical flexural reinforcement of two 1/2" strands and two steel bars, shear span to depth ratio of 45.3"/16.2" = 2.8, and strand embedment length of 55.1". The beams were reinforced with various types of shear reinforcement and designed to fail in shear. The pretensioned beams failed prematurely with strand slippage after shear cracking, and excessive shear cracking and deflection resulted without further reduction of load capacity. The strength of each of the beams was comparable to that calculated according to the ACI prediction for the development length from the point where a large shear crack crossed the prestressing strands. Salmon and McCrate's equation [31] adequately predicted the load at strand slippage for the non-prestressed strands. One of the non-prestressed beams exceeded the

predicted load without strand slippage, and failed in shear as a result of propagation of the diagonal shear crack into the top flange.

2.3 Pullout Tests

In 1961, Keuning [22] reported the pull-out test results of prestressed and untensioned 1/4" strands with embedment lengths in concrete of 1.5", 3", 6", and 9". The results indicated no significant increase in bond strength of the prestressed strand gradually released of its prestress into the concrete in comparison to the pull-out of untensioned strand. The prestressing force was, however, not reported. Only the first of seven series was completed and reported.

In 1970, Stocker and Sozen [35] reported the results of 486 pullout tests of mostly 1" bond lengths, and evaluated on the nature of bond between strand and concrete and the effects of many related variables. The variables investigated were strand diameters (1/4", 3/8", 7/16", and 1/2"), concrete strength (2300 to 7600 psi), shrinkage and settlement of concrete, active confining pressure, and the time of sustained loadings. Anchorage length was theoretically calculated from the bond behavior of 1" bond length, particularly the load-slip relationships, and compared favorably to that measured from 5 beam transfer length tests. Design recommendations include an expression for determining the anchorage length of prestressing strand with coefficients to account for different design parameters.

In 1972, Edwards and Picard [12] obtained the bond-slip relations of six pullout specimens and crack width of tensile bond specimens. Three covers, 0.5", 1.0", and 1.5" were used. The strands were held horizontally during casting. The bond-slip curves could be idealized as elasto-plastic, and there was no deterioration in the maximum bond stress with increasing slip unless a longitudinal crack occurred. For the small number of specimens tested, the shape of the curve was found to be independent of the method of testing, but the maximum bond stress increased as the cover decreased.

In 1981, Burnett and Ahmed [5] described the generalized load-deformation characteristics of untensioned prestressing strands for 3/8" diameter strand. Embedment length, strand size, and concrete (grout) quality were considered important factors. The authors observed that the maximum load occurred at some small displacement after general slip at the loaded end.

In 1984, Scribner and Kobayashi [32] tested 111 specimens for a qualitative study of the effects on bond stress between concrete and untensioned seven-wire prestressing strand, under monotonic and reversed cyclic loadings, of concrete strength, strand size, externally applied lateral confinement, and the magnitude and sequence of application of strand slips. Bond stress was observed to increase with greater slip. The bond lengths in all the specimens were 1" long, and the strand surface was abraded and cleaned. Concrete strength was found to have little effect on bond strength before slip.

After general slip, higher concrete strength contributed to higher bond strength. Strand diameters of 1/4", 7/16", and 1/2" were linearly related with bond strength before initial slip.

In 1988, Laldji and Young [23] examined, both theoretically and experimentally the bond action between prestressing strand and cement grout and the effects of strand type, embedment length, grout strength, and lateral compression. The theoretical approach to the frictional force generated at the interface of cement grout and strand was reasonably confirmed by the experimental results. The experimental results showed proportionate increase in bond stress with lateral compression, and the upper limit to this increase was probably caused by local crushing adjacent to the interface at a lateral compression of 30% of concrete cube strength. The bond is approximately proportional to the square-root of the concrete cube strength. A progressive increase in bond resistance with slip was observed.

In 1989, Griffin [14] investigated the pullout strength of untensioned prestressing strand and the effects of strand diameter, concrete strength, length of embedment, and strand surface condition. Thirty-nine pullout specimens of 6" square in cross-section were embedded concentrically with a straight strand of diameter 1/2" Regular, 1/2" Special, 9/16", or 6/10". A linear relationship was established for embedment length and the steel

stress developed at slip. Surface condition was found to significantly affect the pullout strength.

In 1990, Brearley and Johnston [2] tested 52 pullout specimens with 270-ksi epoxy coated and uncoated strands of 3/8", 1/2" and 6/10" in diameter. The authors concluded that grit-impregnation on epoxy coating of prestressing strand improved pullout bond strength proportionately with the grit density. Pullout bond strengths were numerically between the transfer and flexural bond strengths.

Chapter III

TEST PROGRAM

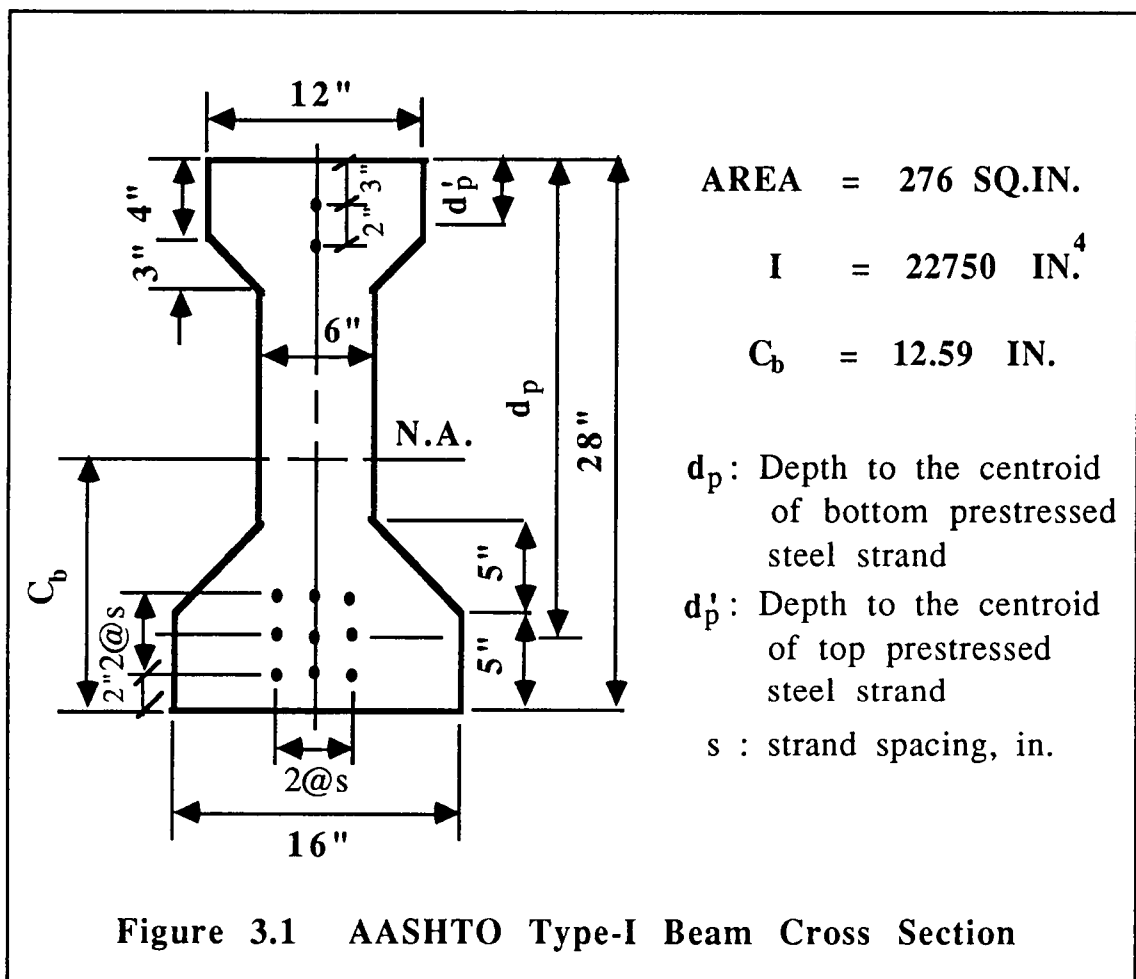
Beginning in late April of 1989, an experimental research project was conducted at The University of Tennessee at Knoxville (UTK) to investigate the bond development of high strength seven-wire prestressing strand for prestressed concrete products. This research project, which primarily involved fabrication and destructive testing of a total of 22 full-scale AASHTO Type-I prestressed concrete bridge beams, was completed in November 1989.

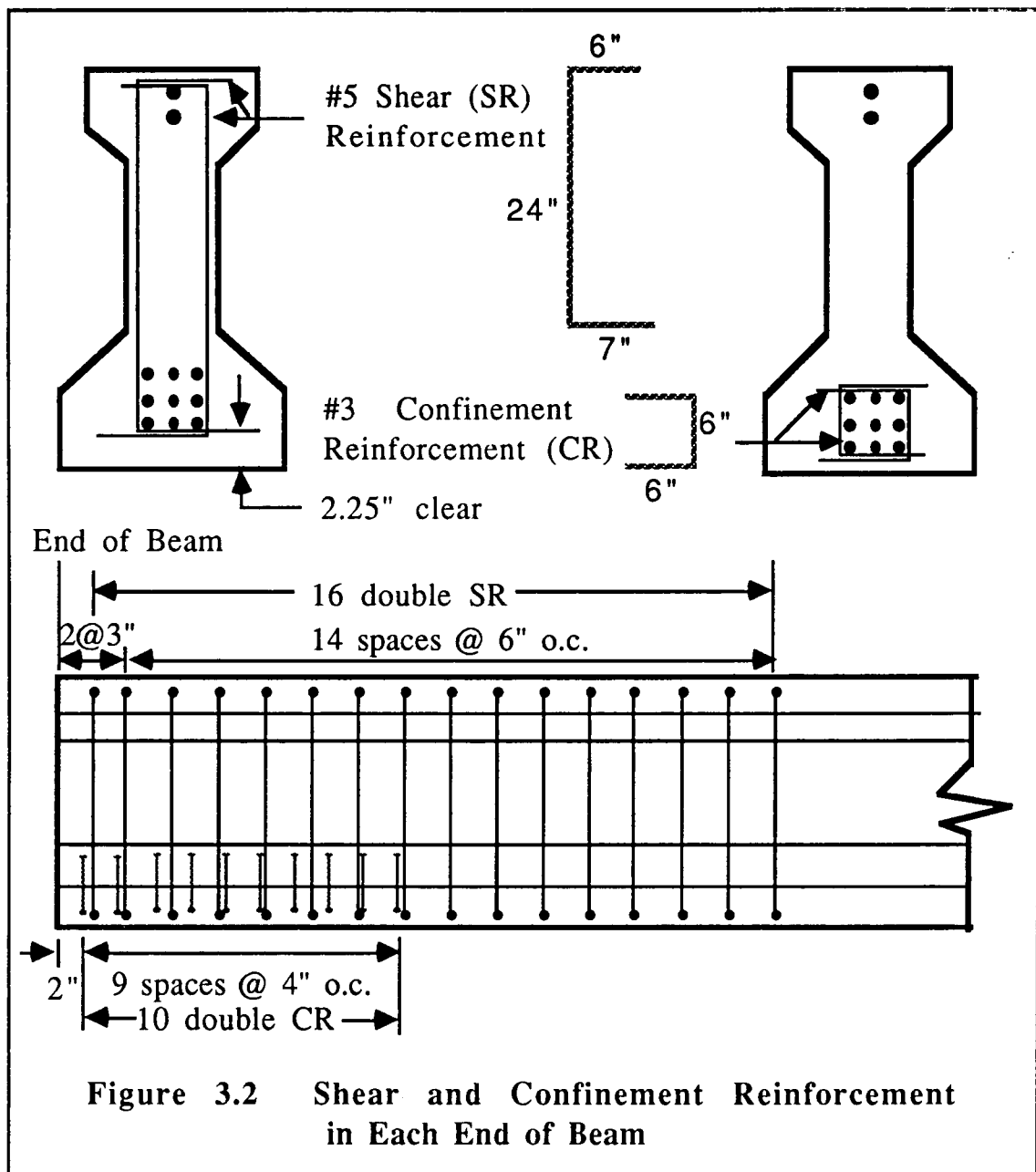
3.1 Specimen Design and Designation

Three types of specimens were made in this study: full-scale prestressed concrete bridge beams, prestressed concrete transfer prisms, and non-prestressed concrete pullout prisms. The bridge beams were tested for transfer length and end slip at transfer during the fabrication process and for development length in static destructive tests. The transfer prisms and the pullout prisms were used to examine the effect of test methods and the effect of strand prestress level on bond development.

3.1.1 Bridge Beams

All beams were 31 feet long, designed to be sufficient in length for two development length tests on each beam. The beam specimens were designed such that each end of the beams provided for a set of strand end-slip measurements, a set of transfer length data, and a development length test. Figure 3.1 illustrates the cross-sectional properties and dimensions of the beams. Figure 3.2 shows details of shear and confinement reinforcement used in all beams.

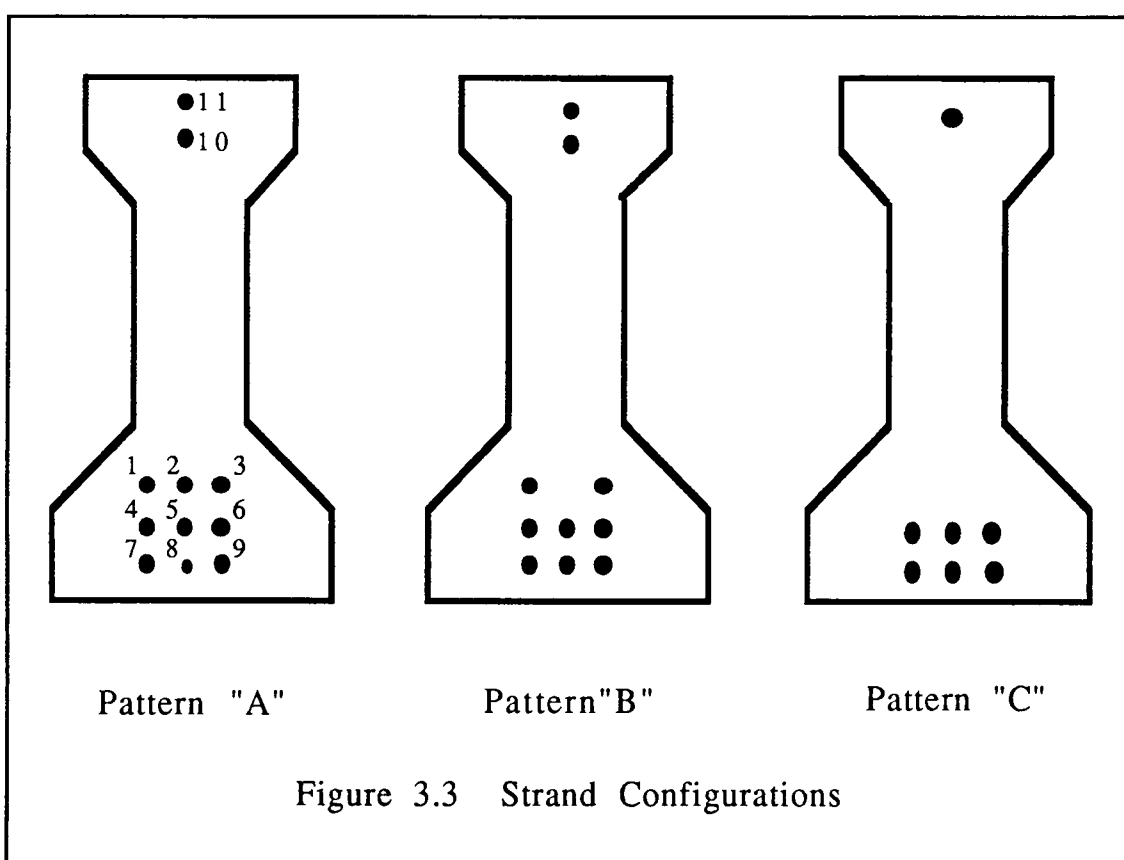




Sixteen of the beams in four strand-diameter groups and the two beams with grit-impregnated epoxy-coated strands all had strands from Florida Wire and Cable Co. (FWC) at a strand spacing of 2". Four beams in each of the strand-diameter groups were prestressed with strands either of 1/2" regular, 1/2" special, 9/16"

or 6/10" in diameter. The other two beams were prestressed with 1/2" epoxy-coated strands. The remaining four beams in a strand manufacturer's group had 1.75" strand spacing, and each was prestressed only with 1/2" regular strands from one of the following manufacturers - Shinko Wire America Inc. (SWAI), Union Wire Rope (UWR), FWC, and American Spring Wire Corp. (ASW).

The initial prestress in the strands of all the beams was designed to be 202 ksi, or 75% of the specified ultimate strength of the strands. The number of prestressing strands used in a beam varied with the size of the strands. Figure 3.3 shows the different strand configurations used.



All uncoated strands were encased in moisture-proof wrapping when received and appeared to be free of rust and shiny when prestressing was applied. The surface condition of these strands is labeled as "Milled". However, several beams had strands that had been exposed to weathering in the casting bed for a few days before the concrete was cast. The durations of weathering for the strands of these beams are noted in Table 3.1 under the surface condition. Table 3.1 also summarizes the strand configurations used for all the beam specimens, including the areas of both the top and the bottom steel strands, and the distances to the centroids of the top and the bottom steel strands from the top of the beam.

A three-part designation is used to identify an end of a beam, as illustrated by "5S-1-EXT". The first part refers to the diameter of the strands used in a beam where:

- 5 = 1/2" Regular strand, nominal diameter of 0.500"
- 5S = 1/2" Special strand, nominal diameter of 0.5224"
- 916 = 9/16" strand, nominal diameter of 0.5625"
- 6 = 6/10" strand, nominal diameter of 0.600"
- 5E = 1/2" epoxy-coated strand.

For the beams from the strand-diameter groups or with the epoxy-coated strands, the second part of the designation refers to one of the beams prestressed with the strands, and the third part refers to a specific end of this beam. The INT or EXT refers to the interior or exterior end of a beam as defined in Figure 3.4. For the beams from the strand manufacturer's group, the second part of the designation

TABLE 3.1 SUMMARY OF BEAM PROPERTIES

SPECIMEN	STRAND	STRAND	STRAND	DEPTH OF STEEL, IN.		STEEL AREA, SQ. IN.	
DESIGNATION	SURFACE	PATTERN	SPACING, IN.	BOTTOM	TOP	BOTTOM	TOP
5-1-EXT	MILLED	B	2.00	24.25	4	1.224	0.306
5-1-INT	MILLED	B	2.00	24.25	4	1.224	0.306
5-2-EXT	MILLED	B	2.00	24.25	4	1.224	0.306
5-2-INT	MILLED	B	2.00	24.25	4	1.224	0.306
5-3-EXT	W-1DAY	B	2.00	24.25	4	1.224	0.306
5-3-INT	W-1DAY	B	2.00	24.25	4	1.224	0.306
5-4-EXT	W-1DAY	B	2.00	24.25	4	1.224	0.306
5-4-INT	W-1DAY	B	2.00	24.25	4	1.224	0.306
5-SWAI-EAST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-SWAI-WEST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-UWR-EAST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-UWR-WEST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-FWC-EAST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-FWC-WEST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-ASW-EAST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5-ASW-WEST	W-3 DAYS	B	1.75	24.47	4	1.224	0.306
5S-1-EXT	MILLED	A	2.00	24.00	4	1.503	0.334
5S-1-INT	MILLED	A	2.00	24.00	4	1.503	0.334
5S-2-EXT	MILLED	A	2.00	24.00	4	1.503	0.334
5S-2-INT	MILLED	A	2.00	24.00	4	1.503	0.334
5S-3-EXT	W-3 DAYS	B	2.00	24.25	4	1.336	0.334
5S-3-INT	W-3 DAYS	B	2.00	24.25	4	1.336	0.334
5S-4-EXT	W-3 DAYS	B	2.00	24.25	4	1.336	0.334
5S-4-INT	W-3 DAYS	B	2.00	24.25	4	1.336	0.334
916-1-EXT	MILLED	B	2.00	24.25	4	1.536	0.384
916-1-INT	MILLED	B	2.00	24.25	4	1.536	0.384
916-2-EXT	MILLED	B	2.00	24.25	4	1.536	0.384
916-2-INT	MILLED	B	2.00	24.25	4	1.536	0.384
916-3-EXT	W-3 DAYS	C	2.00	25.00	3	1.152	0.192
916-3-INT	W-3 DAYS	C	2.00	25.00	3	1.152	0.192
916-4-EXT	W-3 DAYS	C	2.00	25.00	3	1.152	0.192
916-4-INT	W-3 DAYS	C	2.00	25.00	3	1.152	0.192
6-1-EXT	MILLED	C	2.00	25.00	3	1.302	0.215
6-1-INT	MILLED	C	2.00	25.00	3	1.302	0.215
6-2-EXT	MILLED	C	2.00	25.00	3	1.302	0.215
6-2-INT	MILLED	C	2.00	25.00	3	1.302	0.215
6-3-EXT	MILLED	C	2.00	25.00	3	1.302	0.215
6-3-INT	MILLED	C	2.00	25.00	3	1.302	0.215
6-4-EXT	MILLED	C	2.00	25.00	3	1.302	0.215
6-4-INT	MILLED	C	2.00	25.00	3	1.302	0.215
5E-1-EXT	EPOXY	B	2.00	24.25	4	1.224	0.306
5E-1-INT	EPOXY	B	2.00	24.25	4	1.224	0.306
5E-2-EXT	EPOXY	B	2.00	24.25	4	1.224	0.306
5E-2-INT	EPOXY	B	2.00	24.25	4	1.224	0.306

Note: W - Weathered

refers to the name of the manufacturer, and the third part refers to an end of the beam.

3.1.2 Pullout Prisms

Two nonprestressed concrete prisms of 18" and 36" in length were cast along with each beam casting. All the prisms had a square cross-section of six inches in dimension, with a strand embedded longitudinally in the middle of the cross-section. The strand for the prisms was that used for that beam casting.

Each of the prisms provided for a pullout load test in which load versus slip relationship was obtained. The test is designated by the beam cast and the embedment length of the prism, such as "5-1&2-36" for the cast of the beams 5-1 and 5-2, and the prism of 36" length.

3.1.3 Transfer Prisms

In addition to the bridge beams and the pullout prisms, six prestressed concrete transfer prisms were also cast with strands from FWC. Each of the concrete transfer prisms was twelve feet long and prestressed with a single prestressing strand located concentrically through its square cross-section. The cross-sectional area was varied proportionately with the effective prestress force to be transferred to the prism. At each end of the concrete prisms, a set of transfer length data was obtained. Table 3.2 summarizes, for

the prestressed concrete prisms, the cross-section, strand size, and initial prestress in the strand.

Each end of the transfer prisms has a three-part designation as depicted by "5-175-1E". The first part refers to the strand diameter used: "5" for 1/2" strand, "6" for 6/10" strand. The second part refers to the prestress level of the strand in the unit of ksi. The third part refers to the specific end of one of the two prisms cast in a line: "1E" for the exterior end of prism one, "2I" for the interior end of prism two.

Table 3.2 Transfer Prism Specimens			
Specimen Designation	Strand Diameter (in.)	Initial Prestress (ksi)	Cross-Section (in. x in.)
5-175-1E, 5-175-1I 5-175-2E, 5-175-2I	1/2	175	3.5 x 3.5
5-202-1E, 5-202-1I 5-202-2E, 5-202-2I	1/2	202	3.75 x 3.75
6-202-1E, 6-202-1I 6-202-2E, 6-202-2I	6/10	202	4.5 x 4.5

3.2 Materials

All prestressing steel used was seven-wire, low relaxation strand with a specified ultimate tensile strength of 270 ksi. Uncoated strands of four different sizes supplied by FWC were 1/2", 1/2" special, 9/16", and 6/10" diameter strands. Epoxy-coated 1/2" Regular steel strand with aluminum-oxide grit-impregnation was

also provided by FWC. Because of a possible alkali reaction of crushed-glass with concrete, the crushed-glass grit used on the epoxy-coated strands tested by Cousins, et. al. [6,7,8] has been replaced by aluminum-oxide since 1987. Additional 1/2" diameter uncoated strands were furnished by three other manufacturers: SWAI, UWR, and ASW. Nominal diameter (d_b), cross-sectional area (A_{ps}), and pitch of twist of the outer wires of the strands are given in Table 3.3. The pitch of twist is the distance along the length of the strand over which a wire of the strand makes a complete revolution. The diameter of the 1/2" epoxy-coated strand measured over the surface of the coating is 0.58".

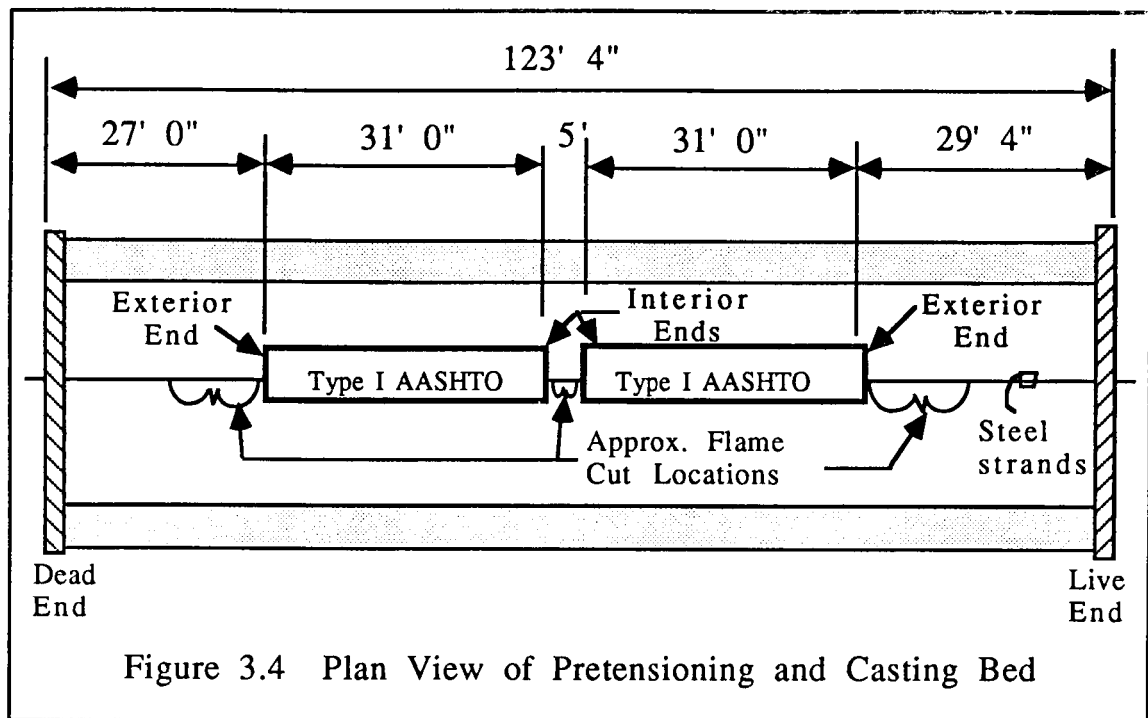
Table 3.3 Properties of Prestressing Strand			
Strand size	Diameter (in.)	Aps (in.)	Pitch of Outer wires (in.)
1/2" Regular	0.5	0.153	7
1/2" Special	0.5224	0.167	7
9/16"	0.5625	0.192	8.38
6/10"	0.6	0.217	9

All shear and confinement reinforcement used in the bridge beams was of ASTM A-615, Grade 60 mild reinforcing steel. A Tennessee Department of Transportation concrete mix design for 28-day concrete strength of 5000 psi was used. The concrete mix design is shown in Table 3.4.

Table 3.4 Concrete Mix Design	
Quantity	Description
752 lb	Type I Cement (Signal Mountain)
1920 lb	No. 67 Coarse-Aggregate (American Limestone Co.)
1326 lb	Fine-Aggregate Manufactured Sand (American Limestone Co.)
21.2 oz.	Low Range Pozzolin 300 (Master Builders)
35 gal.	Water

3.3 Fabrication of Specimens

All test specimens were fabricated by a local producer in accordance with the generally accepted production practices approved by the Precast/Prestressed Concrete Institute. Two beams from each of the strand diameter groups were cast simultaneously, end to end, in a 123'-4" long prestressing bed. Figure 3.4 illustrates a plan-view of the casting bed and defines the exterior and interior faces of each beam. The two beams with epoxy-coated strands were made in one separate cast. The four beams having 1.75" strand spacing were cast individually with strands solely from each of the four manufacturers. In every beam cast, two nonprestressed concrete prisms of 18" and 36" in length were also cast. The six concrete transfer prisms were all made in one final separate cast.



The fabrication process for prestressed specimens generally consists of the following three stages: stressing of strands, casting of specimen, and transferring of prestress force. However, slight modification was made in the strand stressing process for beams that had strands instrumented with electrical strain gages. An initial pretension was applied to all of the strands of these beams before installation of strain gages. After the instrumentation was completed, the strands were later fully stressed to the specified pretension of 202 ksi.

Stressing was accomplished with an electronically controlled hydraulic jack at the live end of the bed. To obtain the correct prestress, the strands were stressed individually to a predetermined force and then checked by comparing the observed elongation to a calculated elongation for the desired prestress in the strands. The

calculated elongation accounted for an estimated loss due to chuck seating.

After the full prestress of the strands was applied, shear and confinement reinforcement for each beam were tied, and the forms were then placed around it. The concrete was mixed on the site and was placed directly into the forms with a three-cubic-yard bucket. The concrete was then vibrated to provide good consolidation around the strands. The pull-out prisms were also cast and vibrated. Eight test cylinders were made for each beam and were used to monitor the concrete compressive strength at prestress transfer and at the static testing of the beam. After placement of the concrete, the casting bed was covered with polyethylene and steam-cured for 8 to 14 hours.

The transfer of the prestress force was made when the concrete compressive strength reached a minimum of 4000 psi, except for two cases where the prestress was transferred when the compressive strengths were 3350 and 3750 psi. Transfer of the prestress force was achieved by simultaneously flame cutting the strands at both ends of the beams one strand at a time. The approximate locations of the flame cuts from each end of the beams were recorded.

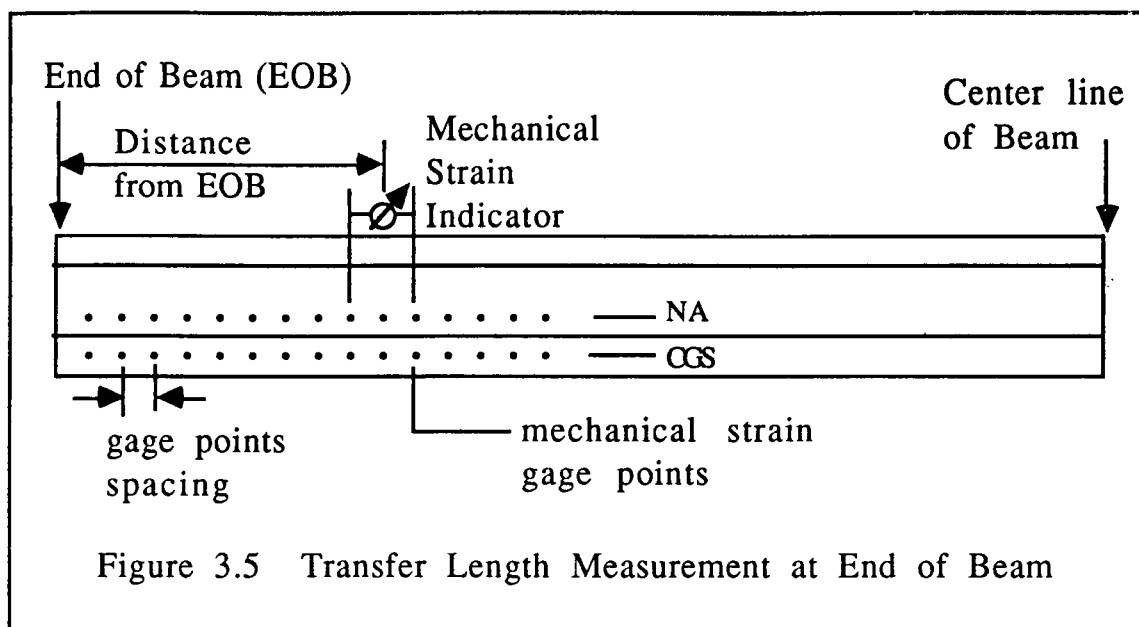
3.4 Instrumentation and Test Procedure

After all the strands in each beam had been pretensioned to 5 kips each in the casting bed, electrical resistance strain gages were

bonded parallel to the longitudinal axis of the helical individual outer-wires of the twisted strands to monitor any changes in strain in the strands. These gages were then waterproofed and mechanically protected. The gages were located from the end of the beam a distance equal to the estimated development length. Steel strains were measured using a portable Vishay/Ellis - 20 digital strain indicator. Strain readings were zeroed before full initial prestress was applied to the strands, and the changes in strain were monitored before casting of concrete, right before and right after prestress transfer, and before the development length test.

3.4.1 Transfer Length Measurements

Transfer length measurements were made on both ends of a beam using the following method. After removing the forms, mechanical gage points were affixed to both sides of the beam, using M-Bond 200 adhesive made by Micro-Measurement Group, Inc. Starting from each end of the beam, 15 gage points were equally spaced at about 4.92" (125 mm). The gage points were located along the neutral axis (NA) and the center of gravity of the steel (CGS), as shown in Figure 3.5. Using a Mayes Mechanical Strain Indicator having a nominal gage length of 9.82" (250 mm) and an accuracy of 0.001" (0.002 mm), the precise lengths between the gage points were measured immediately before and after detensioning of all the strands.



The total deformation on the concrete surface over a gage length is the algebraic difference in two respective readings. The average strain is the total deformation divided by the gage length and is assumed to occur at the middle point of the gage length. The average of the average strains of the corresponding middle points on both sides of the beam is calculated and plotted against the longitudinal distance of the middle points from the end of the beam. Such a plot of transfer length strain distributions is made for the NA and the CGS on each end of the beam. For the concrete transfer prisms, the transfer length strain distribution is plotted the same way, except that the NA is also at the CGS.

3.4.2 End Slip at Transfer

The total slippage of a strand relative to the end surfaces of the beam upon prestress transfer was measured individually at each end

of the beam. First, a piece of adhesive tape was wrapped around each strand at about one inch away from the concrete end surfaces of the beam. Next, a fine mark was made on each tape. Using a dial caliper made by Starrett having an accuracy of 0.001", the precise distance of the mark from the concrete end surface was measured immediately before and after all the strands were cut.

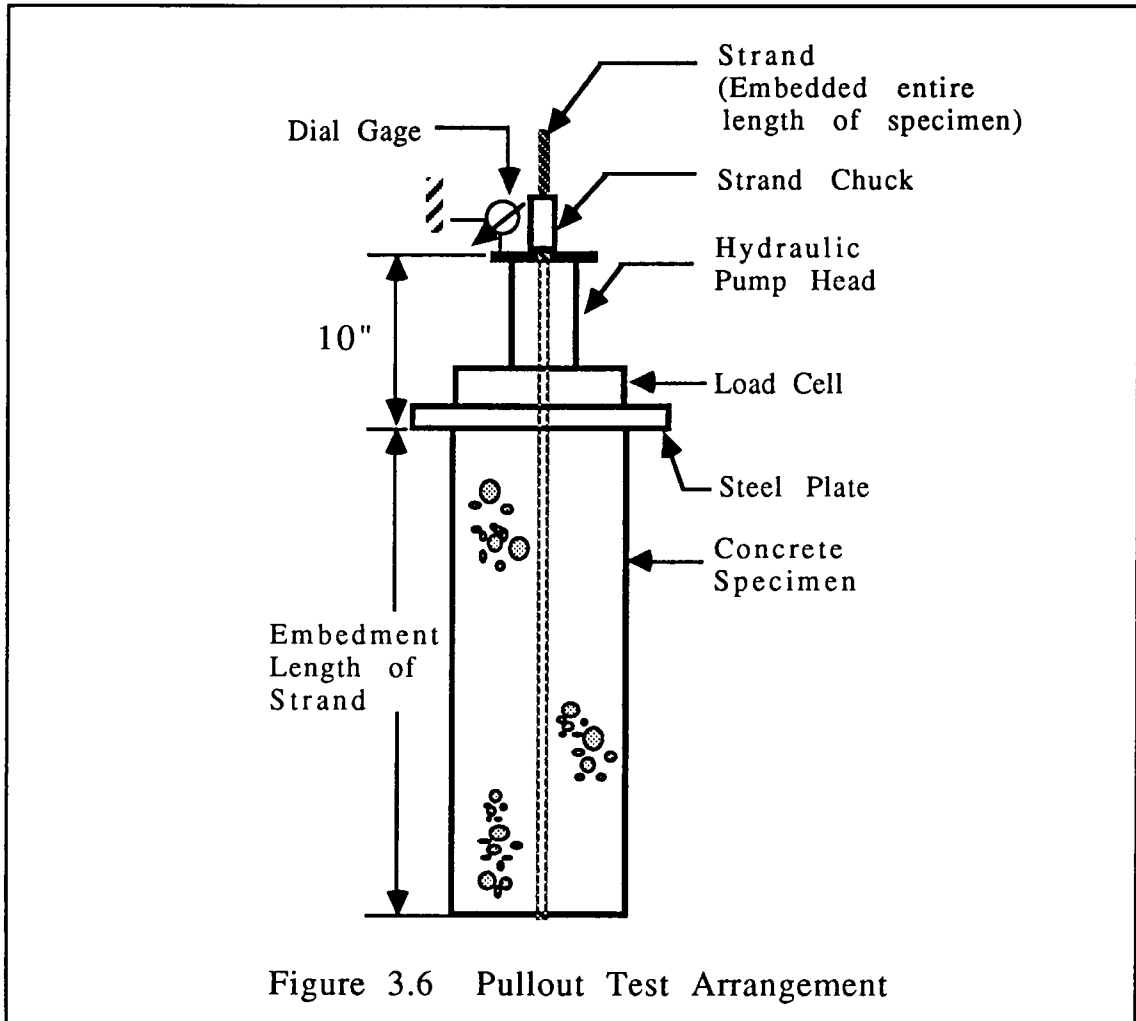
The amount of end slip of each strand is the algebraic difference of the two respective readings adjusted for the axial shortening from prestress release of the strand length between the mark and the concrete surface.

3.4.3 Pullout Tests

A test arrangement as shown in Figure 3.6 was used for the pullout bond tests of nonprestressed strands in concrete prisms. With extension of the hydraulic jack, a reactive force was created from restraint of the strand between the concrete prism and the strand chuck which gripped the strand. The load was measured by the load cell which rested on the 1" thick steel plate that helped to distribute the load onto the concrete. A dial gage measured the deflection of the strand at the strand chuck.

A small load was induced to seat the strand chuck and lightly tighten the strand before resetting the dial gage to monitor the pullout relative slip of the strand. The load was incremented at 0.2 to 1 kip, depending on the length of specimen, until major slippage of the strand had occurred. The dial gage reading was taken at each

load increment. The relative slip at the loaded end is the measured deflection corrected for the axial elongation of the strand of a length of 10" between the face of the concrete and the strand chuck. Load versus deflection relationship for each test is plotted.



3.4.4 Development Length Tests

Bearing 6" at each end onto two supporting steel tubes that were filled with concrete, all of the beams were individually loaded using a hydraulic ram as illustrated in Figure 3.7. The displacement

at the top of the beam at the load point was monitored by a Linear Variable Differential Transformer (LVDT). The displacement measured by the LVDT was used to control the movement of the hydraulic ram head. A load cell attached to the ram head monitored the load on the ram. Dial gages were mounted on all the bottom strands at the loaded end of the beam to measure any strand end slippage during the test. The load at which a slip of as much as 0.001" was detected on any of the strands was recorded manually. The load and displacement at the top of the beam and the steel strains on the strands were read continually into a computer data storage by means of a computer data acquisition system developed for this project. A data file was created for each test to store sufficient data points for later data analysis. Data plots, like a Load versus Deflection curve, could be generated instantly in the laboratory.

Attempts were made to determine iteratively the full development length of the beams. The load was applied on a beam at a distance from the end of beam equal to the estimated full development length for the type of strands in the beam. If end slip (bond failure) of any of the strands occurred before reaching the ultimate flexural compression failure, subsequent tests of the beams with the same strands would have the load located at a longer distance from the end of the beam. Conversely, if flexural compression failure occurred without any strand slippage,

subsequent tests would be with the load at a shorter distance from the end of the beam.

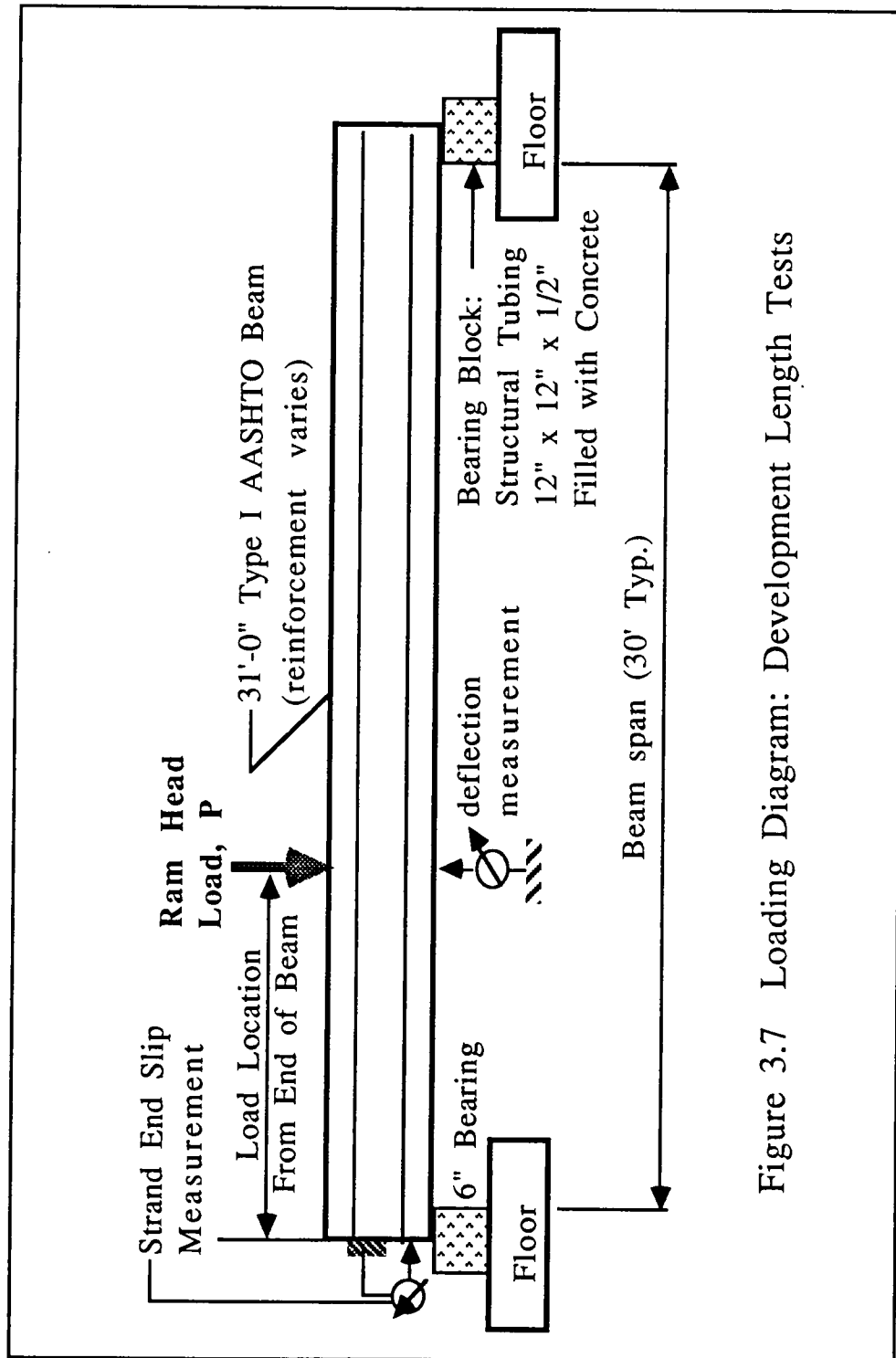


Figure 3.7 Loading Diagram: Development Length Tests

Chapter IV

TEST RESULTS

Considerable experimental data have been gathered in the UTK test program described in Chapter III. At each end of the bridge beams, a set of measurements of transfer length, strand end slip, and steel strain was obtained at strand release during the fabrication process, and load versus deflection and load versus steel strain relationships were acquired during the development length test. The ends of strands were gaged during the test, and any end slips detected were recorded. The ultimate mode of failure for each test, either shear or flexure, was identified. In addition to the bridge beams, load versus slip relationships were obtained from two non-prestressed concrete pull-out prisms of 18" and 36" in length that were cast along with each beam casting. Transfer lengths for strands at different levels of pretension were also measured in a number of concrete prisms, each of which was concentrically prestressed with a strand.

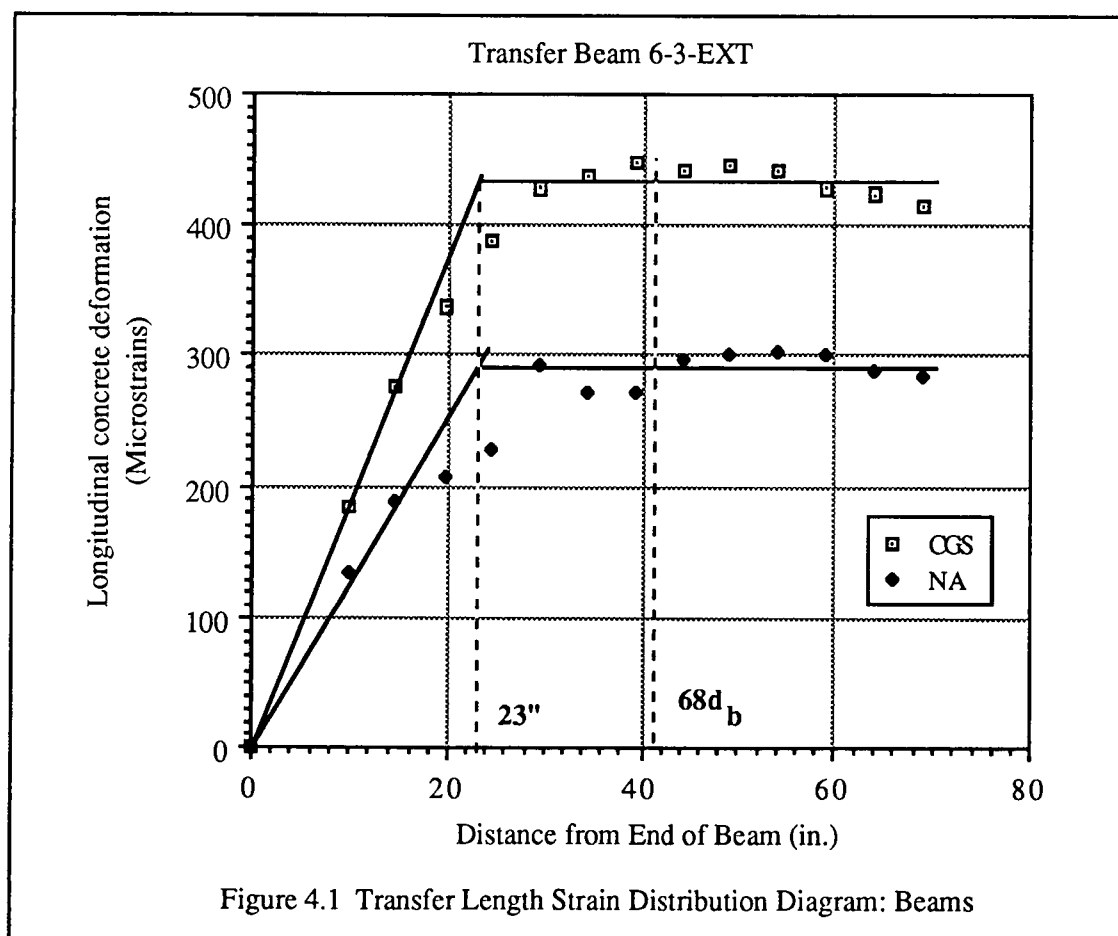
4.1 Bridge Beams

On the total of 22 full-scale AASHTO Type-I prestressed concrete bridge beams, 43 development length tests have been successfully performed. Forty-four sets of transfer length strain distribution data have been obtained. A large number of strand end

slip measurements have also been made at the ends of the beams. These results are presented in the following subsections, complete with full interpretation of each type of data.

4.1.1 Transfer Length

Figure 4.1 presents a typical transfer length concrete strain distribution diagram upon prestress transfer, while the transfer diagrams for all specimens can be found in Appendix B. Two subsets of data points are plotted in the figure. Each data point represents the average of the corresponding strain measurements from both



side-faces of the end of the beam. The subset with the larger strains relates to the data from the CGS, while the other with the smaller strains relates to the data from the NA.

The transfer length is defined as the length over which significant bond resistance exists to stabilize a prestressed strand. This bond resistance exists at the ends of a beam immediately after prestress transfer and is specifically called the transfer bond. Because multiple layers of strands were used in the beam, the measured concrete strain was the average effect of the transfer bond of all the strands. With the concrete strain being linearly proportional to the compressive force at any section for low concrete stress, the average steel stress gradient (or the transfer bond) of the strands can be calculated from the slope of the concrete strain versus transfer length curve in Figure 4.1.

Ideally the strains in the concrete would remain virtually constant beyond the transfer length unless there were transverse loadings. The constant concrete strains imply that a horizontal line can be constructed for the portion of the beam beyond the transfer length. Some random deviations in the data points from the supposedly horizontal portion of the strain distribution diagram are obvious. The deviations are probably due to the nonhomogeneous concrete and the limited precision of the measurements. It is reasonable to assume that such random errors also exist within the transfer length, thereby making the reconstruction of the actual concrete strain distribution diagram there less definitive.

Careful interpretation of the data points obtained is essential to reconstruct the actual strain distribution diagram correctly and determine the transfer length. Previous studies [5,12,22,31,32,35] on the bond characteristics of prestressing strands have consistently demonstrated that the bond behavior of the strands is of the nature of small elasticity and large plasticity. The bond behavior implies that the transfer bond is mostly constant over the transfer length. Therefore, a straight line fitting the data points can be drawn that makes a constant slope in the transfer length portion of the strain distribution diagram. The intersection of this line with the horizontal line previously drawn as the average strain on the concrete with full effective prestress is approximately the end of the transfer bond zone. Due to the nature of the instrument used which measures the average change of strain within a nominal gage length of 9.82", a sharp change in slope of the actual strain distribution diagram near the end of the transfer length gives the average strain measurements there that appear to fit a transitionally smooth curve. The point right at the end of the transfer length had the maximum concrete strain equal to that beyond the transfer bond zone, but the measured average strain over the gage length centered at the point would be less than the maximum. Failure to recognize this fact led Brian Smith [33] to identify transfer lengths from the same data that, on second look, appear to be too large. For example, in one case a transfer length based on the bi-linear relationship of 23 in. was considered to be about 41 in., a significant difference.

The method of data interpretation just presented, which might be termed a *Slope-Intercept* method, is a reasonable and consistent method for determining transfer length. When the results are finally translated to a design equation, a multiplier greater than unity may need to be applied to assure a conservative design.

Table 4.1 summarizes the concrete strength at transfer, f'_{ct} , the specified initial prestress, f_{si} , elastic shortening (E.S.) as measured from the change in strain on the concrete surface of each specimen in the unit of microstrains ($\mu\epsilon$) and the corresponding steel stress loss, Dfs , the effective prestress, f_{se} , and the transfer length, L_t , obtained immediately after transfer according to the Slope-Intercept method. The f_{se} is obtained by subtracting the Dfs from the f_{si} . For comparison with the measured transfer length, the table also lists the transfer length in accordance with the AASHTO provisions, which specify $f_{se}/3$ times the nominal diameter of the strand. The effective prestress multiplied by the area of an individual strand gives the effective prestress force in the strand. The average transfer bond strength, U_t , is calculated by dividing the effective prestress force in the strand by the transfer length.

About half of the electrical strain gages installed on the strands of the beams did perform throughout the test program. The loss of gages was due to the following factors: 1) the unfavorably narrow and cylindrical surfaces of the individual wires of the high-strength steel strands on which the gages were bonded, 2) the large change in strain that the bonded gages were required to withstand, 3) the

TABLE 4.1 TRANSFER DATA SUMMARY

SPECIMEN	fct	fsi	Elastic Shortening			Measured				Bond	
			Mechanical	Electrical	fse	Lt	AASHTO	Measured Lt	Lt	Strength	
DESIGNATION	(psi)	(ksi)	($\mu\epsilon$)	Dfs(ksi)	dfs(ksi)	(ksi)	(in.)	(db)	Lt,(db)	AASHTO Lt	Ut (kip/in.)
5-1-EXT	3780	203	580	16.4	18	186	36	72	62.0	1.16	0.791
5-1-INT	3780	203	550	15.5	18	187	30	60	62.3	0.96	0.954
5-2-EXT	4170	203	525	14.8	18	188	35	70	62.6	1.12	0.820
5-2-INT	4170	203	520	14.7	18	188	29	58	62.6	0.93	0.991
5-3-EXT	4775	203	450	12.7	--	190	23	46	63.3	0.73	1.263
5-3-INT	4775	203	420	11.8	--	191	22	44	63.6	0.69	1.326
5-4-EXT	5235	203	390	11.0	--	192	22	44	63.8	0.69	1.332
5-4-INT	5235	203	390	11.0	--	192	26	52	63.8	0.81	1.127
5-SWAI-EAST	5129	203	340	9.6	11	193	20	40	64.3	0.62	1.476
5-SWAI-WEST	5129	203	330	9.3	11	193	21	42	64.4	0.65	1.408
5-UWR-EAST	5553	203	305	8.6	--	194	21	42	64.6	0.65	1.413
5-UWR-WEST	5553	203	300	8.5	--	194	19	38	64.7	0.59	1.563
5-FWC-EAST	4775	203	380	10.7	--	192	18	36	63.9	0.56	1.630
5-FWC-WEST	4775	203	415	11.7	--	191	18	36	63.6	0.57	1.622
5-ASW-EAST	5154	203	350	9.9	--	193	21	42	64.2	0.65	1.403
5-ASW-WEST	5154	203	365	10.3	--	192	18	36	64.1	0.56	1.634
5S-1-EXT	5340	203	440	12.2	13	190	33	63	63.4	1.00	0.963
5S-1-INT	5340	203	410	11.4	13	191	33	63	63.7	0.99	0.967
5S-2-EXT	4950	203	430	11.9	13	191	34	65	63.5	1.02	0.936
5S-2-INT	4950	203	415	11.5	13	191	30	57	63.7	0.90	1.063
5S-3-EXT	5410	203	430	11.9	12	191	31	59	63.5	0.93	1.027
5S-3-INT	5410	203	455	12.6	12	190	36	69	63.3	1.09	0.881
5S-4-EXT	5300	203	450	12.5	12	190	35	67	63.3	1.06	0.907
5S-4-INT	5300	203	440	12.2	12	190	22	42	63.4	0.66	1.445
916-1-EXT	3360	203	680	19.9	18	183	42	75	60.9	1.23	0.835
916-1-INT	3360	203	640	18.8	18	184	32	57	61.2	0.93	1.102
916-2-EXT	3750	203	580	17.0	18	186	36	64	61.8	1.04	0.989
916-2-INT	3750	203	580	17.0	18	186	28	50	61.8	0.81	1.272
916-3-EXT	5060	203	390	11.4	16	191	30	53	63.7	0.84	1.223
916-3-INT	5060	203	390	11.4	16	191	23	41	63.7	0.64	1.595
916-4-EXT	4950	203	400	11.7	16	191	30	53	63.6	0.84	1.221
916-4-INT	4950	203	390	11.4	16	191	27	48	63.7	0.75	1.359
6-1-EXT	4100	203	450	12.6	19	190	25	42	63.3	0.66	1.634
6-1-INT	4100	203	450	12.6	19	190	27	45	63.3	0.71	1.513
6-2-EXT	4280	203	490	13.7	19	189	30	50	62.9	0.79	1.353
6-2-INT	4280	203	490	13.7	19	189	24	40	62.9	0.64	1.692
6-3-EXT	5230	203	430	12.0	12	191	23	38	63.5	0.60	1.781
6-3-INT	5230	203	405	11.3	12	191	21	35	63.7	0.55	1.958
6-4-EXT	5450	203	450	12.6	12	190	22	37	63.3	0.58	1.856
6-4-INT	5450	203	425	11.9	12	191	23	38	63.5	0.60	1.782
5E-1-EXT	5164	203	355	9.9	16	193	20	40	64.2	0.62	1.473
5E-1-INT	5164	203	340	9.5	16	193	19	38	64.3	0.59	1.554
5E-2-EXT	5412	203	370	10.3	16	192	22	44	64.1	0.69	1.337
5E-2-INT	5412	203	325	9.1	16	193	16	32	64.5	0.50	1.850

NOTE: "W" for surface condition indicates weathering.

severe environment in which they were to be protected, and 4) the difficulty to exercise the care needed in the installation of the many gages with the temperature and humidity in the field and the lack of sufficient time imposed by the working schedule of the precasting plant. This significant loss of gages may cause some doubt as to the reliability of the remaining electrical strain gages that appear to perform satisfactorily. However, the steel stress loss from the time the full prestress is applied to the time of development length test measured by the electrical gages, Δf_s , compares reasonably well to the steel stress loss after transfer, Δf_{st} , measured by the mechanical strain gages. The Δf_s is generally slightly greater than the Δf_{st} , which is to be expected for the fact that additional loss occurs during the time from the prestress transfer to the development length test. More reassuringly, the initial prestress in the strands measured by electrical strain gages was not significantly different from the specified initial prestress of 203 ksi. Steel stresses in the strands are estimated from the measured strains on the individual wires using the appropriate calibration stress-strain curves, which were obtained from tensile tests of strands installed with electrical strain gages.

4.1.2 End Slip at Transfer

Table 4.2 summarizes the amounts of end slip upon prestress transfer of each strand at each end of the beams, the average of the end slips of the bottom strands and of the top strands, and the distance of the flame-cut location from the end faces of the beams.

TABLE 4.2 SUMMARY OF END SLIP RESULTS

Specimen Designation	Flame-cut Distance, in.	Measured Initial End Slip of Strand, in. (See Figure 3.3)									Average Slip (in.)	
		1	2	3	4	5	6	7	8	9	Bottom	Top
5-1-EXT	13.6	0.131	NS	0.103	0.102	0.134	0.097	0.117	0.146	0.111	0.118	0.401
5-1-INT	12.6	0.079	NS	0.138	0.107	0.131	0.124	0.144	0.150	0.116	0.124	0.270
5-2-EXT	12.9	0.061	NS	0.131	0.093	0.132	0.123	0.077	0.129	0.145	0.111	0.373
5-2-INT	47.4	0.081	NS	0.091	0.105	0.148	0.113	0.070	0.118	0.127	0.107	0.479
5-3-EXT	84.0	0.078	NS	0.061	0.096	0.061	0.080	0.088	0.066	0.069	0.075	0.143
5-3-INT	30.0	0.081	NS	0.065	0.079	0.074	0.077	0.072	0.057	0.091	0.075	0.208
5-4-EXT	60.0	0.060	NS	0.043	0.089	0.052	0.028	0.044	0.052	0.056	0.053	0.113
5-4-INT	30.0	0.099	NS	0.102	0.110	0.073	0.074	0.107	0.106	0.106	0.097	0.215
5-SWAI-EAST	NA	0.043	NS	0.071	0.043	0.073	0.058	0.072	0.084	0.073	0.065	0.167
5-SWAI-WEST	NA	0.063	NS	0.083	0.046	0.058	0.067	0.094	0.087	0.079	0.072	0.129
5-UWR-EAST	NA	0.035	NS	0.071	0.051	0.081	0.072	0.097	0.029	0.031	0.058	0.160
5-UWR-WEST	NA	0.063	NS	0.053	0.023	0.075	0.064	0.051	0.036	0.065	0.054	0.185
5-FWC-EAST	NA	0.065	NS	0.060	0.082	0.057	0.039	0.027	0.070	0.079	0.060	0.111
5-FWC-WEST	NA	0.044	NS	0.042	0.033	0.004	0.006	0.070	0.116	0.132	0.056	0.129
5-ASW-EAST	NA	0.083	NS	0.031	0.028	0.055	0.047	0.103	0.109	0.043	0.062	0.174
5-ASW-WEST	NA	0.031	NS	0.042	0.084	0.034	0.023	0.085	0.059	0.047	0.051	0.127
5S-1-EXT	6.0	0.107	0.097	0.147	0.065	0.080	0.078	0.138	0.122	0.124	0.106	0.247
5S-1-INT	6.5	0.105	0.082	0.089	0.040	0.237	0.361	0.055	0.148	0.023	0.127	0.241
5S-2-EXT	6.0	0.118	0.111	0.139	0.123	0.074	0.071	0.110	0.127	0.138	0.112	0.235
5S-2-INT	54.0	0.071	0.055	0.127	0.077	0.131	0.116	NA	0.180	0.131	0.111	0.200
5S-3-EXT	3.0	NA	NS	NA	0.098	NA	NA	0.082	0.121	0.111	0.103	NA
5S-3-INT	5.4	0.134	NS	0.161	0.123	0.108	0.162	0.165	0.136	0.140	0.141	0.252
5S-4-EXT	9.4	0.150	NS	0.096	0.136	0.108	0.142	0.118	0.126	0.103	0.122	0.341
5S-4-INT	54.6	0.088	NS	0.180	0.059	0.072	0.070	0.070	0.080	0.133	0.094	0.315
916-1-EXT	12.0	0.109	NS	0.134	0.126	0.136	0.124	0.126	0.178	0.133	0.133	0.330
916-1-INT	12.0	0.119	NS	0.067	0.129	0.084	0.088	0.137	0.104	0.097	0.103	0.338
916-2-EXT	12.0	0.098	NS	0.036	0.088	0.093	0.057	0.139	0.084	0.074	0.084	0.284
916-2-INT	48.0	0.098	NS	0.099	0.065	0.066	0.078	0.074	0.111	0.155	0.093	0.313
916-3-EXT	36.0	NS	NS	NS	0.067	0.100	0.105	0.140	0.090	0.094	0.099	0.148
916-3-INT	22.0	NS	NS	NS	0.074	0.043	0.060	0.109	0.078	0.102	0.078	0.202
916-4-EXT	24.0	NS	NS	NS	0.113	0.100	0.075	0.105	0.127	0.116	0.106	0.200
916-4-INT	38.0	NS	NS	NS	0.062	0.060	0.063	0.068	0.081	0.065	0.067	0.189
6-1-EXT	120.0	NS	NS	NS	0.068	0.056	0.074	0.051	0.104	0.098	0.075	0.259
6-1-INT	18.0	NS	NS	NS	0.091	0.079	0.103	0.069	0.082	0.084	0.085	0.263
6-2-EXT	72.0	NS	NS	NS	0.095	0.085	0.094	0.056	0.089	0.062	0.080	0.357
6-2-INT	42.0	NS	NS	NS	0.072	0.081	0.083	0.076	0.049	0.073	0.072	0.368
6-3-EXT	80.0	NS	NS	NS	0.064	0.065	0.052	0.082	0.098	0.046	0.068	0.158
6-3-INT	30.0	NS	NS	NS	0.057	0.056	0.042	0.066	0.062	0.066	0.058	0.129
6-4-EXT	50.0	NS	NS	NS	0.031	0.050	0.049	0.053	0.030	0.068	0.047	0.147
6-4-INT	30.0	NS	NS	NS	0.071	0.081	0.062	0.058	0.050	0.067	0.065	0.180
5E-1-EXT	NA	0.059	NS	0.060	0.092	0.068	0.071	0.059	0.071	0.043	0.065	NA
5E-1-INT	NA	0.017	NS	0.024	0.016	0.027	0.031	0.011	0.025	0.012	0.020	NA
5E-2-EXT	NA	0.076	NS	0.060	0.091	0.081	0.079	0.065	0.065	0.065	0.073	NA
5E-2-INT	NA	0.022	NS	0.040	0.039	0.030	0.030	0.026	0.023	0.037	0.031	NA

Notes: NS - No Strand, NA - Data Not Available

4.1.3 Development Length Tests

The development length test results for all the beams are summarized and presented in Table 4.3. For each test, these figures are tabulated: the concrete strength at the time of test, f'_c , the point from the nearer end of the beam where load is applied, L_p , the location of the electrical strain gages from the end of the beam, L_g , the maximum measured load, P_u , and the corresponding shear force V_u , the mode of failure of the beam, and the measured flexural moments at the load point and steel stresses at the gage point at different important stages during the test.

The flexural moments at the load point are calculated from the measured applied load on each beam with the assumption of simple supports at the ends of the beam. The cracking moment, M_{cr} , is on the average about 66% of the ultimate moment, M_u . The moment at which the first strand slippage is detected, M_b , is also calculated for beams with bond failure.

The effective prestress, f_{se} , in the strands was measured with the electrical strain gages just before each development length test. During the development length test, the observed steel stresses at the gage point at first slippage of the strands, f_{sb} , and at the maximum applied load, f_{su} , are listed. Some higher steel stresses in the strands were also observed in a few of the beams when the beams were deflected beyond their ultimate loads and plastic hinges were forming beneath the load points. These maximum steel stresses observed are tabulated as " f_{sm} ."

TABLE 4.3 DEVELOPMENT LENGTH TEST RESULTS

Specimen Designation	f_c (psi)	Load Location Lp (in.)	Strain gage Location (in.)	Ultimate Load Pu (kip)	Mode of Failure	Max. Shear Vu (kip)	Shear Moment Mer Mb (kip-ft)	f se (ksi)	f sb (ksi)	f su (ksi)	fsm Load (ksi)	Load @ Strand Slippage (kip)	Strand # (Fig. 3.3)
5-1-EXT	5476	92	92	114.0	B-F	91.2	396	611	645	191	259	265	268 108[1]; 110[6,7,8,9]
5-1-INT	5476	69.6	69.6	125.0	B-S	107.0	469	550	564	191	234	240	- 122[1,6,7,8,9]
5-2-EXT	6746	77.4	77.4	124.0	B-S	104.0	444	612	612	191	239	259	262 124[1,3,5,7,8,9]
5-2-INT	6746	85	58	120.0	B-F	98.1	415	623	639	191	234	242	257 117[5,7,8,9]
5-3-EXT	6858	85.1	93	127.0	F	103.0	388	-	675	191	-	-	No Slip
5-3-INT	6858	77.4	81	138.0	B-F	115.0	408	645	679	191	262	265	270 131[7,8]; 135[9]; 138[1,3,5]
5-4-EXT	7600	81.25	67	132.0	F	109.0	410	-	676	191	260	260	- No Slip
5-4-INT	7600	77.4	86	135.0	B-F	112.0	453	644	664	191	N/A	-	- 131[N/A]
5-SW1-EAST	5553	81	84	125.3	F	104.0	430	-	641	200	-	269	273 No Slip
5-SW1-WEST	5553	69.6	87	147.0	F-B	125.0	485	660	660	200	269	270	274 147[N/A]
5-UWR-EAST	5989	77.4	-	138.0	B-F	115.0	463	630	679	-	-	-	- 128[N/A]
5-UWR-WEST(S)	5989	69.6	-	177.0	B-F	134.0	459	611	707	-	-	-	- 153[N/A]
5-FWC-EAST	5341	72.5	-	137.5	B-F	117.0	419	624	640	-	-	-	- 134[N/A]
5-FWC-WEST	5341	77.4	-	134.0	F	112.0	448	-	660	-	-	-	- No Slip
5-ASW-EAST	5400	73.5	-	134.3	F	114.0	429	-	633	-	-	-	- No Slip
5-ASW-WEST	5400	69.6	-	142.0	F	121.0	463	-	638	-	-	-	- No Slip
5S-1-EXT	6624	69	69	127.0	F	109.0	466	565	569	199	232	232	- 126[4]
5S-1-INT(LC)	6624	81	50	142.0	F-B	117.0	449	719	724	199	217	217	- 141[7,8,9]; 138.5[4]
5S-2-EXT	6624	84	90	142.0	F	117.0	449	719	724	199	217	217	- 141[7,8,9]; 138.5[4]
5S-2-INT(S)	6800	82.5	82.5	161.0	F-B	100.0	408	692	692	199	260	260	- 161[N/A]
5S-3-EXT	5967	81	66	125.0	F-B	103.0	471	-	640	200	-	253	261 125[1,4,6,7,8,9]
5S-3-INT	5967	75	75	122.0	B-S	103.0	457	587	587	200	245	245	- 122[1,4,6,7,8,9]
5S-4-EXT	6181	68	66	130.0	B-S	112.0	442	530	574	200	241	243	- 120[1]; 130[5]
5S-4-INT	6181	72	111	144.0	B-F	122.0	472	601	666	200	241	255	- 130[5]; 144[7,8]
916-1-EXT	5533	106	106	110.0	B-F	83.9	488	682	688	185	240	240	248 109[7,8]
916-1-INT	5533	87	87	130.0	B-F	105.0	476	633	703	185	222	242	258 117[1]; 122[7,8]
916-2-EXT	5921	96	65	114.0	B-F	90.0	453	642	665	185	210	-	- 110[7,8,9]; 112[4,6]; 114[1,3]
916-2-INT	5921	87	87	126.0	B-F	102.0	465	677	682	185	N/A	203	- 125[1,3,4,6,8,9]
916-3-EXT	6119	95.5	114	114.0	B-F	90.1	419	628	663	187	220	218	- 95[7]; 108[8]; 110[9]; 113[4]
916-3-INT	6119	104.4	105	109.0	F	83.7	390	-	675	187	-	-	- No Slip
916-4-EXT	6237	104.4	87.5	108.0	B-F	82.9	390	601	669	187	222	246	248 97[N/A]
916-4-INT(LC)	6237	108	95.5	108.0	F	81.9	429	-	688	187	-	251	- No Slip
6-1-EXT	5126	116	116	99.0	F	73.2	412	-	658	184	232	-	- No Slip
6-1-INT(S)	5126	93	66	155.0	F	95.3	335	-	682	184	217	-	- No Slip
6-2-EXT	5285	83.5	83.5	126.0	B-F	103.0	398	660	660	184	232	-	- 126[N/A]
6-2-INT	5285	74.4	92	138.0	B-F	116.0	462	609	657	184	226	244	- 128[N/A]
6-3-EXT	7463	83.52	85.5	134.0	B-F	110.0	424	696	701	191	N/A	-	- 133[N/A]
6-3-INT	7463	88.16	93	129.0	F	104.0	426	-	704	191	-	268	- No Slip
6-4-EXT	7984	85.84	89	130.0	F	106.0	406	-	695	191	-	262	- No Slip
6-4-INT	7984	85.84	95.5	133.0	F	108.0	428	-	711	191	-	265	- No Slip
5E-1-EXT	5783	58.08	78.00	180.0	F	158.0	436	-	683	193	-	270	272 No Slip
5E-1-INT	5783	69.66	55.00	148.0	F	126.0	449	-	665	193	-	260	- No Slip
5E-2-EXT	6573	46.4	65.00	203.0	B-S-F	185.0	382	565	620	193	254	269	271 185[N/A]
5E-2-INT	6573	51.1	69.50	194.0	F	174.0	417	-	652	193	-	269	274 No Slip

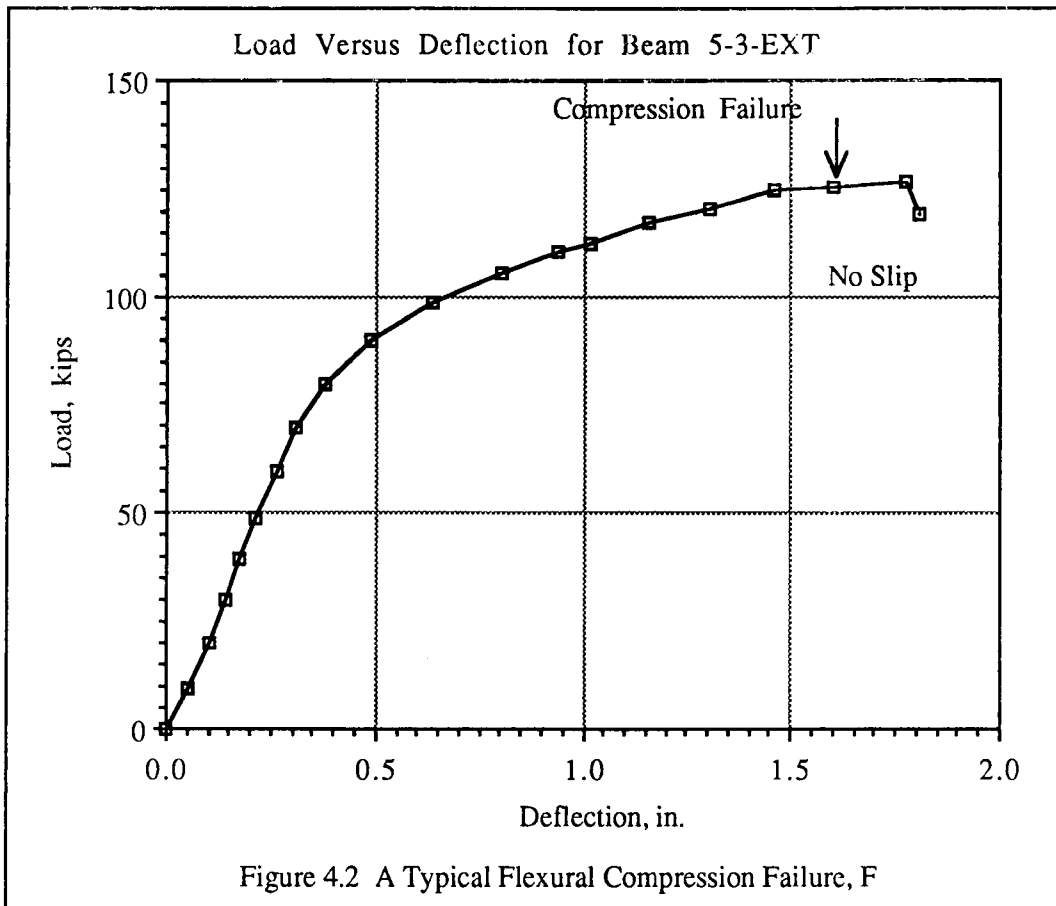
NOTES: B - Bond Failure (Strands Slip), S - Shear Failure, F - Flexural Compression Failure,
(S) - Short Portion of Broken Beam, (LC) - Loading on beam being Load Controlled

Due to a malfunction in the controller of the hydraulic loading equipment, data on the very first development length test was unfortunately not acquired. Of all the 43 development length tests that have been successfully performed, end slip was not detected in 16 tests, four tests showed "slight" end slips after flexural compression failures were developed, 17 tests experienced end slips but did eventually fail in flexural compression, five tests suffer shear failure immediately after a bond failure, and one test with epoxy-coated strands was loaded very close to support that it failed in shear-compression with a bond failure.

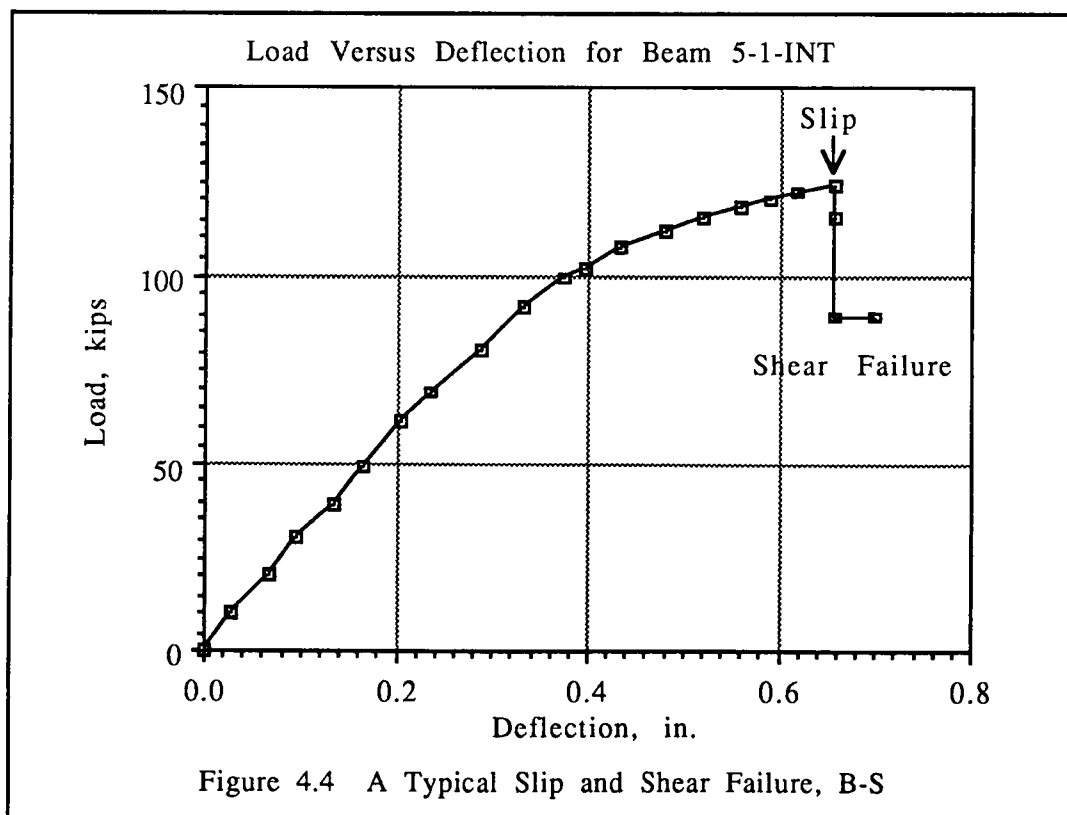
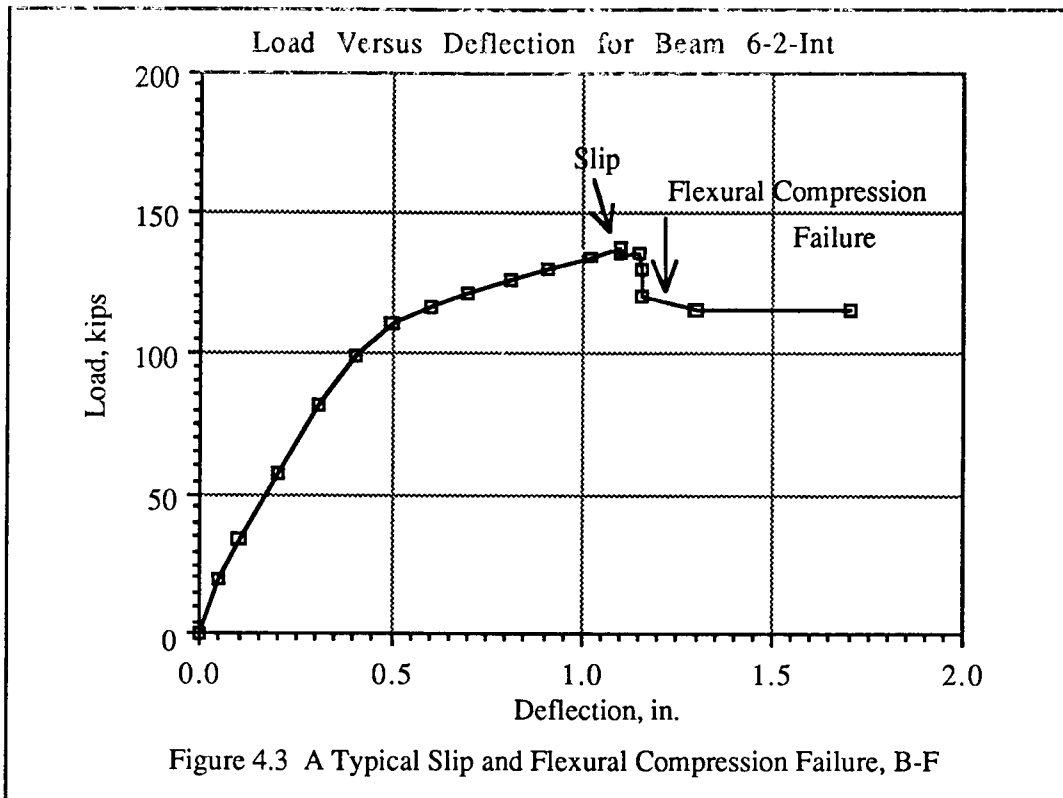
Beam Behavior and Mode of Failure

The behavior of the beams can be generally categorized into four different modes of failure, which are explained briefly here with reference to the typical load versus deflection plots. The load versus deflection plots for all beam tests can be found in Appendix C.

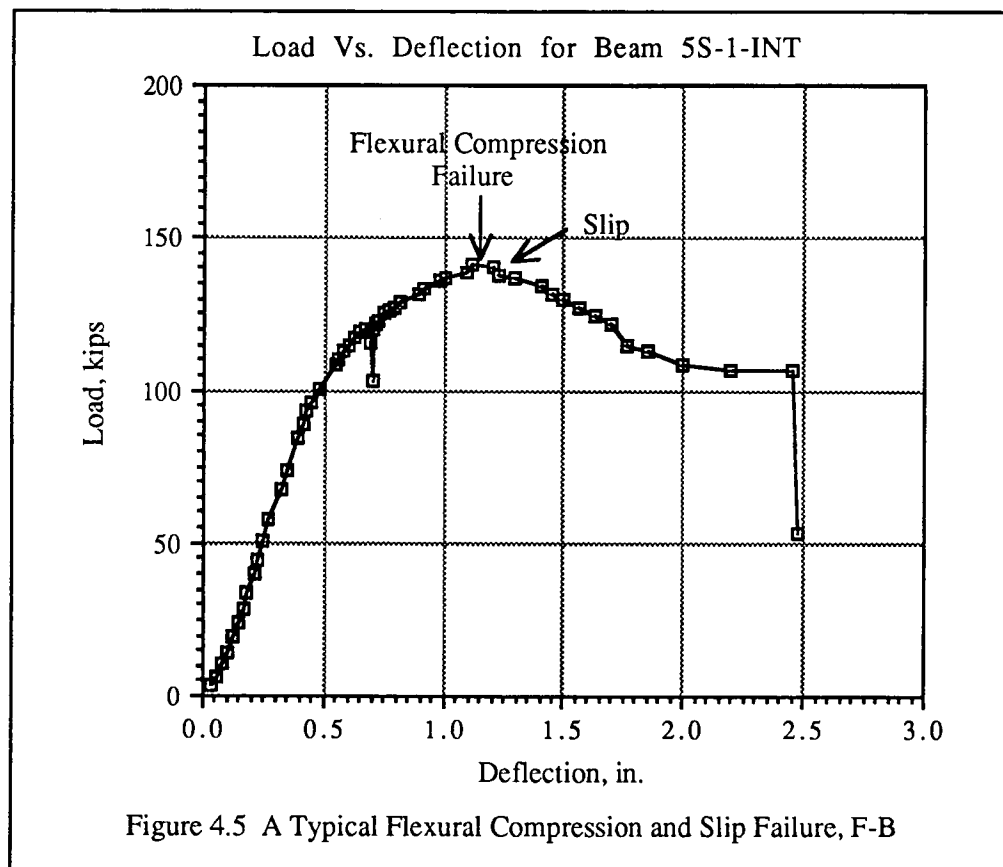
Figure 4.2 shows a typical load versus deflection plot for a flexural failure denoted in Table 4.3 by "F". This plot is for test 5-3-EXT. The beam exhibited linear behavior up to cracking; then the stiffness rapidly decreased with increasing deflection and ultimately concrete crushing at the maximum moment. The failure is typical of a beam with adequate development length and thus without any strand slippage.



The typical load versus deflection plots for tests with bond failure are shown in Figure 4.3 and Figure 4.4. The beams showed the typical linear behavior up to cracking load and then nonlinear behavior until a bond failure indicated by end slip. The bond failure occurred prematurely before the ultimate flexural strength. Losses in bond were subsequently accompanied by premature flexural compression failure or shear failure, denoted as "B-F" or "B-S" respectively.

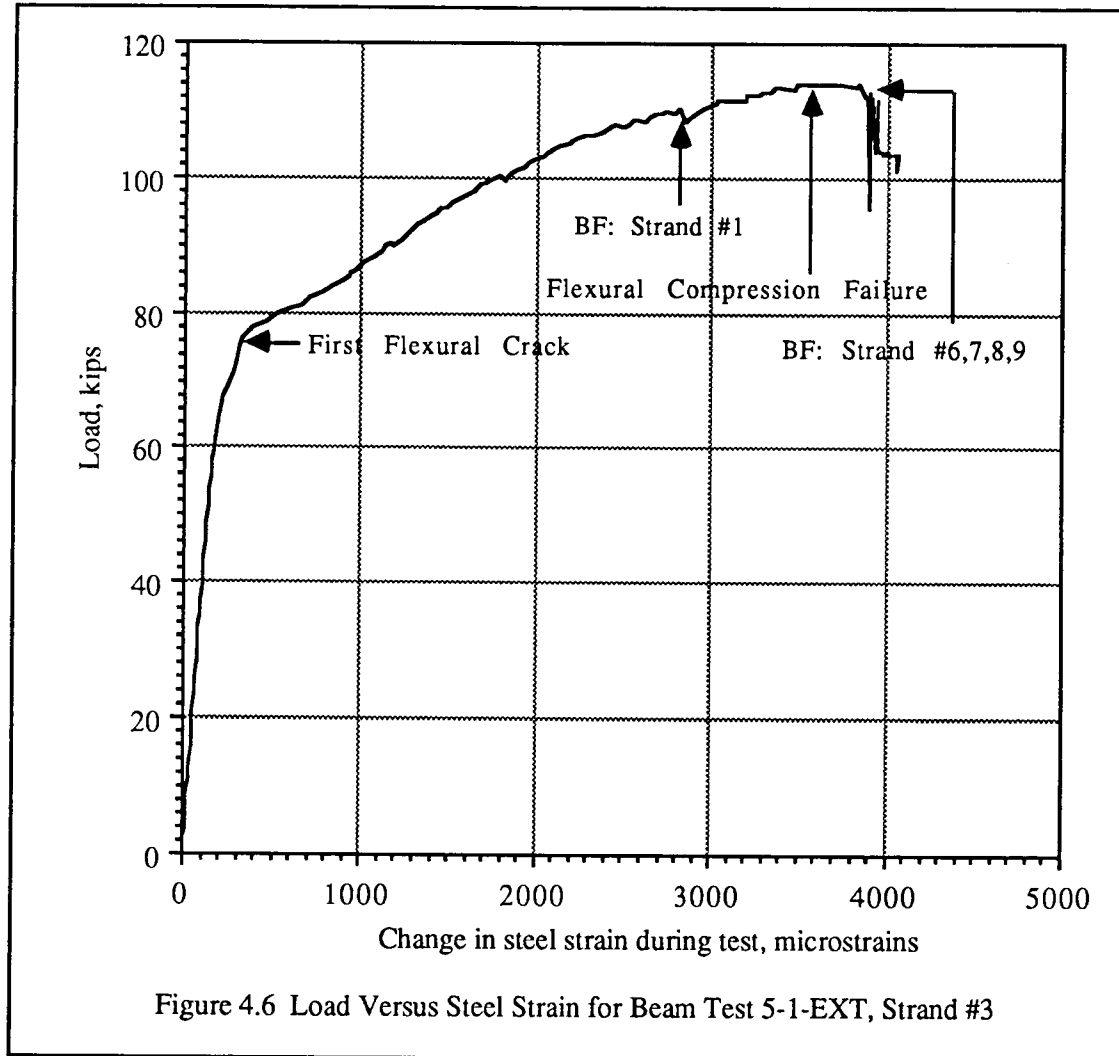


When a bond failure is only detected very near to the flexural compression failure or just after the compressed concrete fiber begins to crush, the beam is evidently loaded at a point very close to the full development length. This mode of failure is then denoted "F-B", and a typical load versus deflection plot for this is in Figure 4.5.



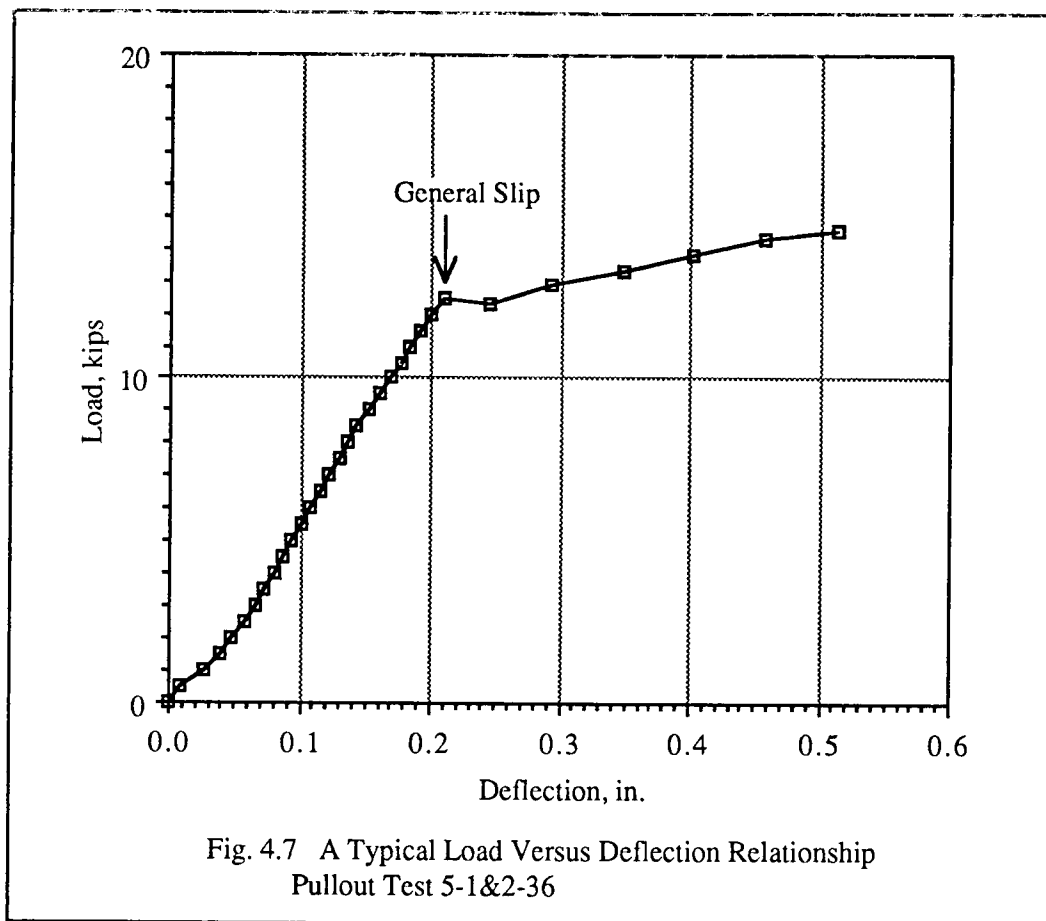
Stress increase in the steel above the effective prestress is relatively small up to cracking of the section. These stress ranges, however, are typical of service conditions because service loads in pretensioned slabs and beams are usually below the cracking loads. As indicated by the strain gages, steel strains below the load point increased abruptly at the initial flexural crack and continued to

increase until the failure of the beam. Figure 4.6 shows a measured load versus steel stress plot that clearly illustrates this fact.



4.2 Pullout Prisms

Figure 4.7 shows a typical load versus deflection relationship for a pullout test. The plots for all the pullout tests can be found in Appendix D. The measured deflection is the total of the relative slip at the loaded end and the axial elongation of the strand of a length of



10" between the face of the concrete and the strand chuck. In a monotonic loading, there is a load above which significant deflection occurs without much increase in load. This is noted in the figure as the point of general slip, the point at which relative slip between the strand and the surrounding concrete occurs over the entire embedment length of the strand. However, the criterion for general slip is limited only to strands that have relatively short embedment lengths. There is a length above which a strand would yield in tension before the general slip of the strand is reached; test 5E-1&2-36 may be such a case. The rapid increase in deflection for the test is probably contributed by the axial elongation of the yielding strand.

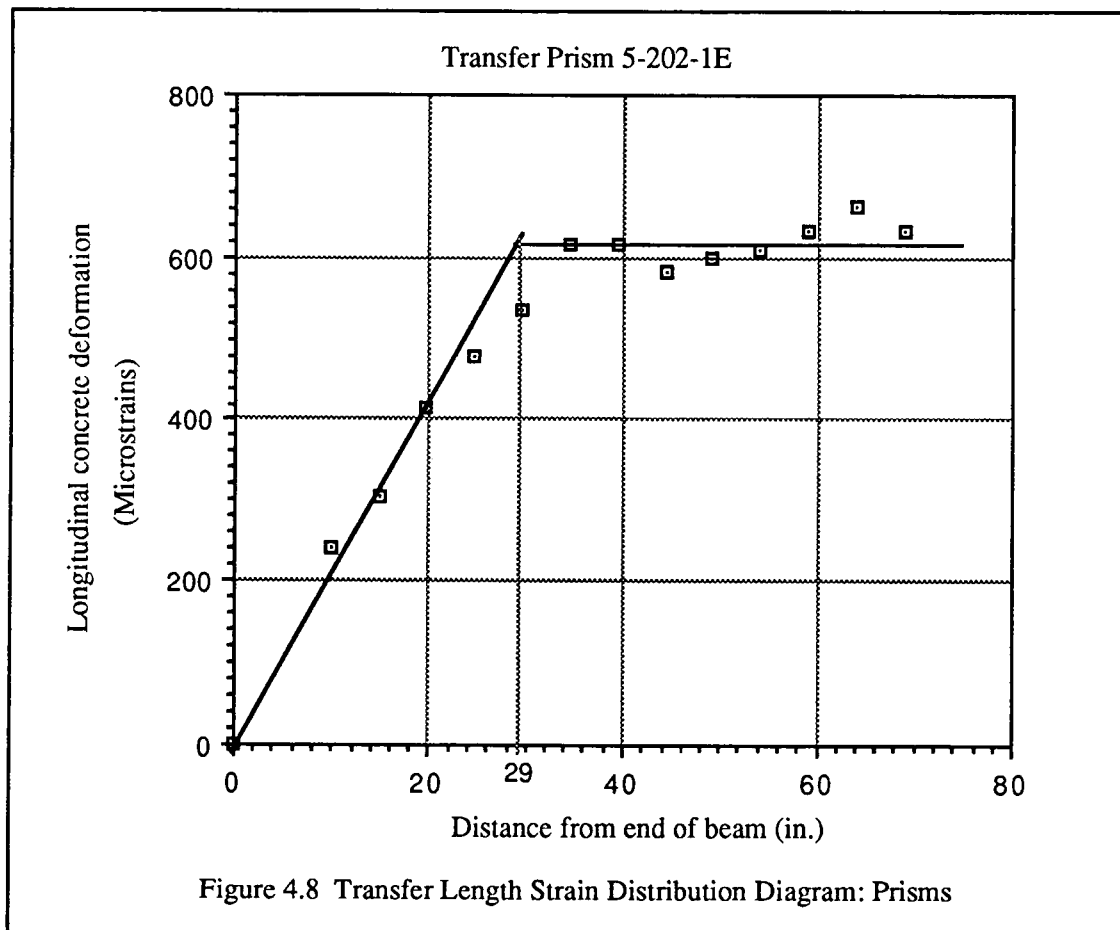
Table 4.4 summarizes for each pullout prism the concrete strength at the time of testing, f'_c , the surface condition of the strand when the prism was cast, the load that causes a general slip, and the average bond strength at the general slip, U_p . The average bond strength is the load listed divided by the embedment length.

TABLE 4.4 PULLOUT TEST RESULTS				
Specimen Designation	f'_c (psi)	Surface Condition	Measured Load at Slip (kips)	Bond Strength U_p (kips/in.)
5-1&2-36	5500	Milled	12.60	0.350
5-3&4-18	6370	Milled	7.40	0.411
5-3&4-36	7070	Milled	17.50	0.486
5-SWAI-18	5530	W - 3 days	17.00	0.944
5-SWAI-36	5530	W - 3 days	35.00	0.972
5-UWR-18	5989	W - 3 days	18.00	1.000
5-FWC-18	5341	W - 3 days	13.50	0.750
5-ASW-18	5400	W - 3 days	18.70	1.039
5S-3&4-18	5360	Milled	7.20	0.400
5S-3&4-36	5360	Milled	11.50	0.319
916-1&2-18	3560	Milled	10.20	0.567
916-1&2-36	3560	Milled	11.50	0.319
916-3&4-18	5010	Milled	9.00	0.500
916-3&4-36	5010	Milled	14.50	0.403
6-1&2-18	4190	Milled	10.40	0.578
6-1&2-36	4190	Milled	24.00	0.667
6-3&4-18	5340	Milled	13.80	0.767
6-3&4-36	5340	Milled	29.00	0.806
5E-1&2-18	5160	Epoxy	35.00	1.944
5E-1&2-36	5410	Epoxy	40.00	1.111
Note: "W - 3 days" indicates weathering for 3 days				

4.3 Transfer Prisms

Figure 4.8 shows a typical transfer length strain distribution diagram for a concrete transfer prism. The diagrams for all the transfer prism tests are included in the end of Appendix B. For each

end of the prisms there is only one set of strain measurements, which is the average of the corresponding strain measurements on both sides of the prisms, along the centroid of the steel strand that is also the neutral axis of the prism. Unlike the bridge beams with their multiple strands, the measured concrete strain was the effect only of the single strand in each prism.



Using the *Slope-Intercept* method of reconstructing the actual transfer length strain distribution diagram as explained in Subsection 4.1.1, the transfer lengths are determined for the prestressed prisms and summarized in Table 4.5. Also listed in the table are the axial

elastic shortening (E.S.) in the concrete with the unit of microstrains ($\mu\epsilon$) and the stress loss in the steel due to the elastic shortening. By dividing the effective force in each strand by the transfer length, the average transfer bond strength, U_t , is obtained and tabulated. Transfer length according to AASHTO equation of $f_{se}/3$ times the strand diameter is calculated and compared to the measured transfer length. The strand was cut by flame at one end, and the prism was allowed to slide and relax the strand at the other end. The two ends are respectively called the cut end and the dead end. The transfer prisms with the smallest cross-section of 3.5" x 3.5" cracked at the ends upon prestress transfer even though the initial prestress level of the strand in the prisms was the lower 175 ksi. The concrete strength at transfer was 4760 psi for all the specimens, and all the strands were of milled surface condition.

TABLE 4.5 TRANSFER PRISM TEST RESULTS									
Prism	End	Elastic Shortening		fse	Measured	AASHTO	Measured	Bond	
Designation	(D/C)	($\mu\epsilon$)	(ksi)	(ksi)	Lt. (in.)	Lt. (in.)	AASHTO Lt	Remark	Strength (kips/in.)
5-175-1E	D	510	14.4	161	25	26.77	0.93	CR 5-in.	1.229
5-175-1I	C	500	14.1	161	28	26.82	1.04	CR 5-in.	0.985
5-175-2E	D	575	16.2	159	29	26.46	1.10	CR 3-in.	0.934
5-175-2I	C	580	16.4	159	27	26.44	1.02	CR 5-in.	1.103
5-202-1E	C	615	17.3	185	30	30.86	0.97	-	0.944
5-202-1I	D	600	16.9	186	26	30.93	0.84	-	1.092
5-202-2E	D	640	18.0	184	21	30.74	0.68	-	1.344
5-202-2I	D	600	16.9	186	26	30.93	0.84	-	1.092
6-202-1E	D	600	16.9	186	20	30.93	0.65	-	2.014
6-202-1I	D	600	16.9	186	19	30.93	0.61	-	2.120
6-202-2E	C	600	16.9	186	30	30.93	0.97	-	1.342
6-202-2I	D	640	18.0	184	20	30.74	0.65	-	2.001
Notes: All specimens are 12 ft long, All Milled Strand Surface Condition									
fct = 4760 psi END : D--DEAD, C--CUT, CR--Prism Cracked at End									

Chapter V

DISCUSSION OF TEST RESULTS

The results obtained in the UTK test program and presented in parts in the previous chapter are interrelated. Transfer length has a definite relationship with the amount of end slip of the strand; the relationship is a function of the bond distribution along the transfer length. Development length generally consists of a transfer length and a flexural bond length, and is related to the general beam behavior and strength. In this chapter, the factors that affect the bond development, the influence of the variables on the transfer and development lengths, and the interrelationship of beam behavior and development length are discussed.

5.1 Effects of Variables on Bond Development

Bond resistance that exists between a prestressing strand and the concrete surrounding the strand is attributed to three distinct and independent features of interfacial stress transfer mechanisms: adhesion, friction, and mechanical. In this section, the relationships of these three factors and the phases of the bond resistance to the bond development along the length of a strand are analyzed. Furthermore, the effects of many physical variables on the three factors and, consequently, the development length of a strand are discussed.

Good adhesion can exist probably only on the surfaces of the outer wires of a strand. The strength of adhesion bond is a function of many variables: the surface condition of strand, the quality of cement paste (which includes settlement, shrinkage and strength of concrete), degree of consolidation of concrete, and the normal stress at the concrete-steel interface. The normal stress may be induced by shrinkage of concrete, relaxation of strands or externally applied confinement. However, practical significance of adhesion bond in a strand may be small because the adhesion between the wires and the concrete can be broken at a very small relative interfacial deformation. The small allowable deformation limits the length of adhesion bond that can exist along a strand with stress gradient. The length of adhesion bond is negligibly short within the transfer length of a strand. In a flexural bond length, adhesion bond exists throughout its length before any cracking of the beam. After cracking, the adhesion bond is progressively broken away from a crack while the strand slips to accomodate the participation of the other two bonds.

At the instance the adhesion is overcome and strands begin to slip, friction bond is believed to be mainly responsible for bond action, with increasing resistance provided by mechanical bond at greater relative slips. Friction bond depends on strand surface condition and the normal force in shear surfaces. Small dislodged sand particles in the shear surface also provide some wedging action which adds to sliding resistance. In transfer lengths, the normal

force is created mostly from concrete restraint to swelling of the strands that are relaxed of its prestress. In flexural bond length, the normal force is induced from shrinkage of concrete, but the normal force is relieved by the shrinking of strands from increased applied tension.

Plain concrete around a strand can sustain a relatively small normal force before it cracks. This small normal force, which may be created by swelling of relatively low level of prestress at transfer, may not significantly increase with the swelling at transfer of highly prestressed strand that causes the concrete around a strand to form extensive cohesive cracks. Future research may seek to prove this hypothesis, which may be the reason for the fact that transfer bond strength is independent of the level of prestress, as observed in tests [20] conducted with strands at relatively high prestress. Externally applied forces, like support reactions, offer some effects of confinement which prevent the concrete cracking and add to friction resistance. Confinement reinforcement can slightly increase the stiffness of concrete restraint around a strand before the cracking of the concrete. When the concrete cracks, confinement reinforcement becomes most effective. The effectiveness of confinement reinforcement should increase with more strands within the confinement reinforcement because of greater cumulative strain created by more strands.

Mechanical bond is due to non-uniform sizes of groove space between the outer wires of a strand and the nature of helical

wrapping of the outer wires. The effectiveness and responsive participation of mechanical bond increases with good consolidation of concrete, which eliminates air pockets around a strand and fully packs the concrete into the groove space. Thus, good consolidation of concrete during casting of a beam is extremely important to the bond strength along a strand. The mechanical bond is created after strand slippage when the concrete from larger grooves wedges into smaller grooves. The wedging concrete may be sheared at much greater slip, causing a significant loss of the component of mechanical bond; then, interlocking of the sheared concrete with other concrete may occur to sustain additional bond. Helical wrapping may contribute slightly to friction bond from the bearing force on the sloping surface of wires in a strand. Perhaps helical wrapping also increases the non-uniformity of groove sizes and thus increases the mechanical bond slightly. Stocker and Sozen [35] tested straight seven-wire strands and twisted seven-wire strands and found insignificant difference in the bond strengths of the two types of strands. In transfer length, mechanical bond exists mostly near the free end where strand slippage is the greatest. In flexural bond length, mechanical bond exists mostly near the critical section away from the free end of a strand.

From the general discussion on the features of the three factors of bond resistance that may exist along the development length of a strand, the friction bond and mechanical bond together are considered to be mainly responsible for the steel stress developed in

a strand. The quantity of each of the two bonds in transfer length and flexural bond length may be a subject for future research. However, because transfer bond is mostly composed of both mechanical and friction bonds and pullout bond is mostly the effect of mechanical bond, it may be that the friction bond is roughly the difference in the transfer bond strength and the pullout bond strength.

5.2 Transfer Length Tests

Transfer lengths of the strands were measured at prestress transfer on the bridge beams and small concrete prisms. Table 5.1 lists the average transfer bond strengths for all milled surface strands. For the 1/2" Regular strands in the prism tests, the bond strength for the initial pretension of 175 ksi is only about 3% lower than that for the initial pretension of 202.5 ksi. This small difference in bond strengths indicates a negligible effect of the additional strand pretension on transfer bond strength.

In a comparison of the transfer bond strengths of 1/2" Regular and 6/10" strands from the beam tests with those from the prism tests which had strand pretension of 202.5 ksi, the effects of test methods on transfer bond strength can be seen. The bond strengths of both the strands from the prism tests are apparently higher than those from the beam tests. Confinement reinforcement was placed over a distance of about 40" from each end of the beams, the distance of which is longer than almost all the measured transfer

lengths. In some exploratory tests with spiral smooth-wire confinement reinforcement around each strand, Kaar, LaFraugh and Mass [20] found a decrease of 15% in transfer length as compared to those without the reinforcement. Accordingly, transfer bond strengths for beam tests should be higher. However, other factors such as the multiple strands in beams may have influence on the bond strength. Multiple strands in beams were flame-cut one at a time, and strands cut earlier might have been affected by the transfer force of strands cut later. This effect should degrade transfer bond strength in beams, and perhaps moreso for beams with more strands. From Table 5.1, the decrease in the measured bond strengths for 1/2" Regular and 6/10" strands were about 25% and 10%, respectively. Greater reduction in bond strengths of beams with the 1/2" Regular strands may be due to the greater number of strands in the beams. The strands in the dead ends of prisms were released of prestress more gradually; those ends of prisms also showed a higher bond strength.

Table 5.1 Measured Transfer Bond Strength for Beams and Prisms			
Milled Surface Strand Diameter (in)	Average Transfer Bond Strength (kips/in)		
	Beams fsi =202.5 ksi	Prisms	
		fsi =202.5 ksi	fsi = 175 ksi
1/2" Regular	0.889	1.12	1.08
1/2" Special	0.982	-	-
9/16"	1.050	-	-
6/10"	1.700	1.87	-

Effects of Strand Diameter

The perimeter of seven-wire prestressing strand is approximately equal to $4/3\pi d_b$. Adhesion force, which is directly proportional to the amount of adhered surface, is therefore directly proportional to strand diameter. Friction may be affected by strand diameter due to the difference in normal force from different wire sizes. Because the grooves between the outer wires of a strand get larger with increasing strand diameter, mechanical bond strength would increase with strand diameter. Table 5.2 illustrates that the average transfer lengths for the 1/2" Special, 1/2" Regular, and 9/16" strands of milled surface condition are approximately proportional to the strand diameter, but not the 6/10" strands. The shorter transfer length for 6/10" strands may be attributed to the increase in mechanical bond.

Table 5.2 Average Measured Transfer Lengths of Beams			
Strand	Average Transfer Length (db)		
	Milled	Weather 1-day	Weather 3-day
1/2" Regular	64.5	46.5	39.0
1/2" Special	62.2	--	59.3
9/16"	62.2	--	48.9
6/10"	40.6	--	--

Effects of Strand Surface Condition

The wires of all stress relieved and low relaxation strands have residual surface oil from the wire drawing process. These residuals prevent good adhesion and lower the coefficient of friction at shear

interface, thereby decreasing friction bonds. Weathered strands have their residual surface oil washed away and have roughened rusted surfaces with higher coefficients of friction, which are also better for concrete adhesion. From Table 5.2, the average improvement in the transfer length for one-day weathering of 1/2" Regular strands was about 27.9%, while that for 3-day weathering for the same strand appears to be about 40%. The average improvement for all the 3-day weathered strands was about 22%. This reduction in transfer length may be due to an increase in friction for the slightly roughened surface.

Effects of Concrete Strength

Concrete strength should affect the adhesion bond strength between the concrete and steel, but the effect of adhesion bond strength on bond length is negligible. Friction should not be affected by concrete strength. Mechanical interlock should be dependent on concrete strength because failure of mechanical bond involves shearing of concrete in the grooves between the wires of a strand. As seen in Table 5.2, the greatly reduced transfer length for 6/10" strands may be attributed to the greater mechanical bond strength developed by the concrete in the larger grooves between the wires of the strands.

5.3 Development Length Tests

Development length of strand in a pretensioned member consists partly of transfer length and an additional bonded length

called flexural bond length. Development length, unlike transfer length, cannot be measured directly but only with an indirect but realistic approach in which the behavior and strength of beams loaded at different locations from the ends of beams are studied to determine whether the ultimate stress of the strands could be developed at the load points. This procedure was followed for different strand diameters, surface conditions, concrete strengths, and pretension levels. Different spacings of strands from different manufacturers were used. Blended in the results there are other variables such as concrete strength, reinforcement ratio, and strand pretension level, all of which have some influence on both the behavior and strength of the beam section.

5.3.1 Interrelation of Development Length with Beam Behavior

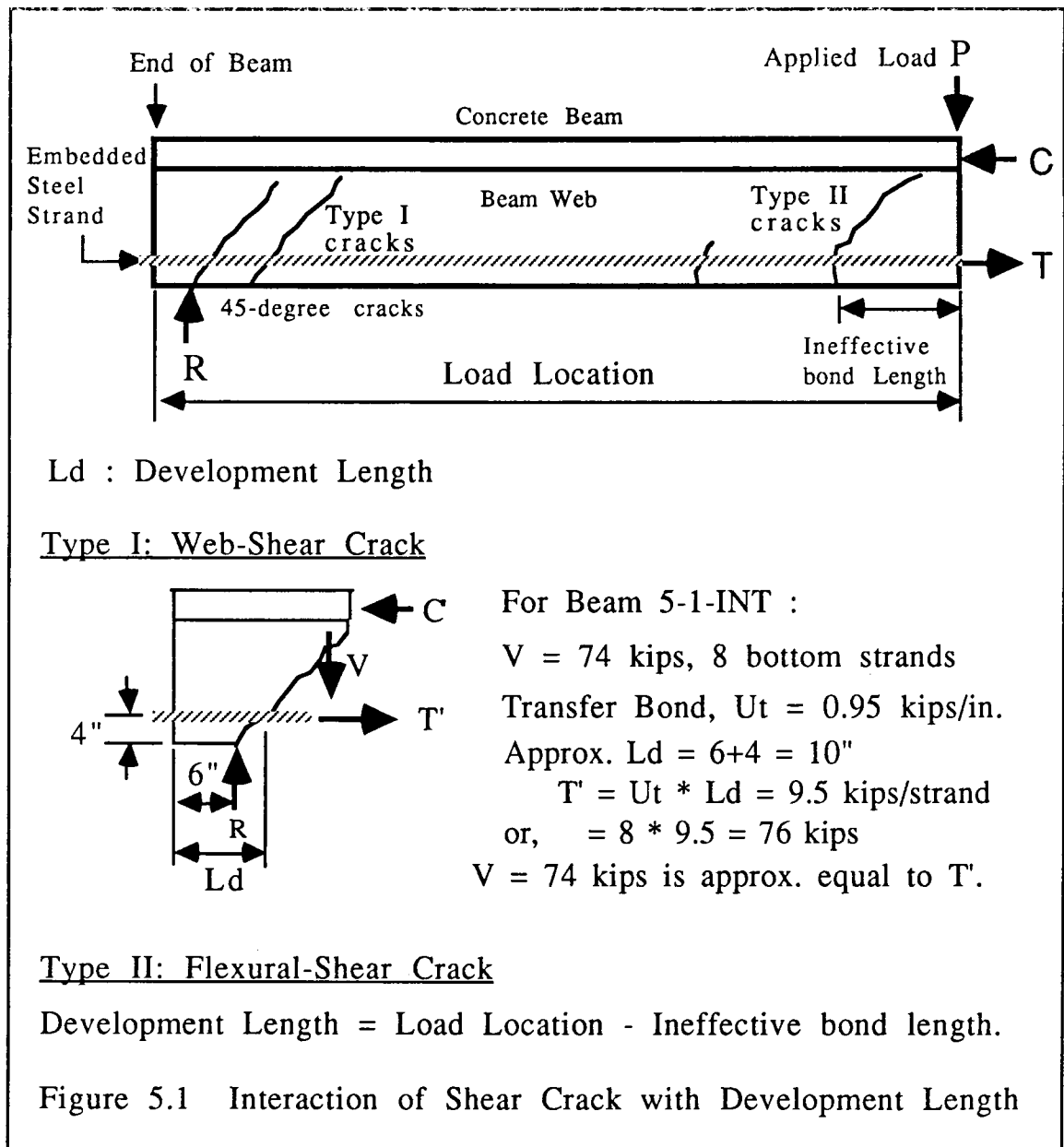
The development length for a beam is interrelated with the beam behavior. The steel stress that a strand can develop at a beam section is affected by variables such as concrete strength, steel pretension level, reinforcement percentage, concrete confinement, and beam geometry. The corresponding force in the strand must also be adequately transmitted along the development length into the concrete. Transmission of the force over the available development length is dependent upon the bond behavior and the beam behavior. For example, the deformational characteristics of the strand at the beam section when the steel force is transmitted over the

development length affects the steel stress that can be developed at the section. Using the procedure described in Appendix A3 to obtain the steel stress developed, the effects on the steel stress of the variables that affect beam strength is minimized.

For beams with lower reinforcement ratios, slight slippage of some strands will only redistribute the stresses among the strands and not immediately result in a lower flexural strength. Thus, a steel stress higher than that at first slippage may be achieved for an underreinforced beam. However, steel in underreinforced beams is generally stressed more than steel in overreinforced beams. Although bond failure is more likely for an underreinforced beam, the flexural strength of the beam may not be affected by the bond failure. On the other hand, flexural strength of overreinforced beams is controlled by the available area of compressive concrete. The overreinforced beams therefore cannot tolerate slippage which reduces the compressive concrete area. These relationships are studied for a better understanding of the effects of strand slippage on beam behavior.

Changes in beam stiffness with different modes of failure have been discussed in the comparison of the load versus deflection plots in the previous chapter. The different modes of failure observed in the destructive tests are examined in detail, particularly the interaction between the flexural compression failure or shear failure with a bond failure. As illustrated in Figure 5.1, a sudden shear failure at the initiation of diagonal shear cracking (type I failure)

may occur without an adequate development length from the end of a beam to the shear crack which starts at the interior face of support. The measured shear load, V , that beam 5-1-INT could sustain after the diagonal cracking was much reduced, i.e. only 74 kips. The effects of the diagonal shear cracks on the bond development seem to be adequately analyzed using the *Truss Analogy* method. The total tensile force (T') that can be developed at the shear crack that initiates from the face of support is calculated from the transfer bond strength for the number of bottom strands. For a 45-degree angle of the shear crack, the shear force V at the cracked section that failed because of insufficient development length would be close to the T' . Also illustrated in the figure is a flexural-shear failure (type II failure) with crack that propagates from a flexural crack some distance closer to the end of beam. This type II failure reduces the flexural bond length of a portion of ineffective bond length that is approximately equal to a distance about the depth of bottom steel. The crackings from type I and type II failure in essence move the critical section for the evaluation of development length to the cracks. Indeed, the interrelation of beam behavior with development length is complex.



5.3.2 Effects of Bond Failure on Beam Behavior

The effects of bond failure on the behavior of all the bridge beams tested are examined in detail and discussed with illustrations. As has also been observed by previous investigators, many of the beams tested could develop additional beam strength even after end

slips of some strands were detected. Bond failure of the strands apparently did not cause a significant loss in the steel stress of the strands, and the additional flexural strength gained was probably contributed by the other strands that did not slip. The plastic nature of the bond behavior permits the strands to keep slipping with virtually no loss of bond resistance. This is an extremely important characteristic of the strand performance in contrast to the plain wires. Slippage of the strands that have exhausted their "usable" bond strength is beneficial because it permits the other strands, not yet slipping and having some reserved bond strength, to still participate in carrying the extra load. Ideally, for a beam that has less than the full development length, maximum flexural strength is developed only when all usable bond strength in all the strands had been exhausted. However, the inability of slipping strands to carry any additional tension reduces the stiffness of the beam. For a beam approaching its ultimate strength with the steel stressed past the yield point and the extreme fiber of compressive concrete stressed into the inelastic range, this loss of stiffness certainly would immediately result in a flexural compression failure.

Research [32,35] with pullout bond tests has suggested that the bond resistance, although with significant variability, generally drops slightly when the last of the adhesive bond is broken at the initiation of strand end-slip, and that higher bond resistance is attainable at greater slip when mechanical bond takes effect. Therefore, bond length has an effect on the drop in load at the loaded end when end

slip at the free end occurs. Stocker and Sozen [35] observed from their test results that the mechanical bond may become effective immediately after slip or only after a relatively large slip of about 0.01". The large slip cannot be accommodated at a flexural section without excessively reducing the compressive concrete area and causing a failure. When only a small proportion of the strands suffer "slight" slips, there may be just a minor drop in load with the stiffness provided by the other strands. Creep of the concrete surrounding a strand that experiences the high intensity of bond stress has been observed in pullout tests to gradually relieve bond resistance and permit "slight" slippage of the strand to occur until it becomes stabilized. This slight slippage probably occurs for the strands in the UTK beams. The load versus deflection plots in Figure 5.2 and Figure 5.3 show that a slight drop in load occurs while the end slips of a small number of the strands are detected. The load capacity of the beams did not reduce with increasing applied deflection until flexural compression failure occurred. However, when most of the strands in a beam slip nearly at the same time, the load drops off significantly not to regain strength and finally fail in a flexural compression failure at increasing deflection; Figure 5.4 shows the load versus deflection plot of test 6-2-INT which has five out of a total of six strands slipping when load drops.

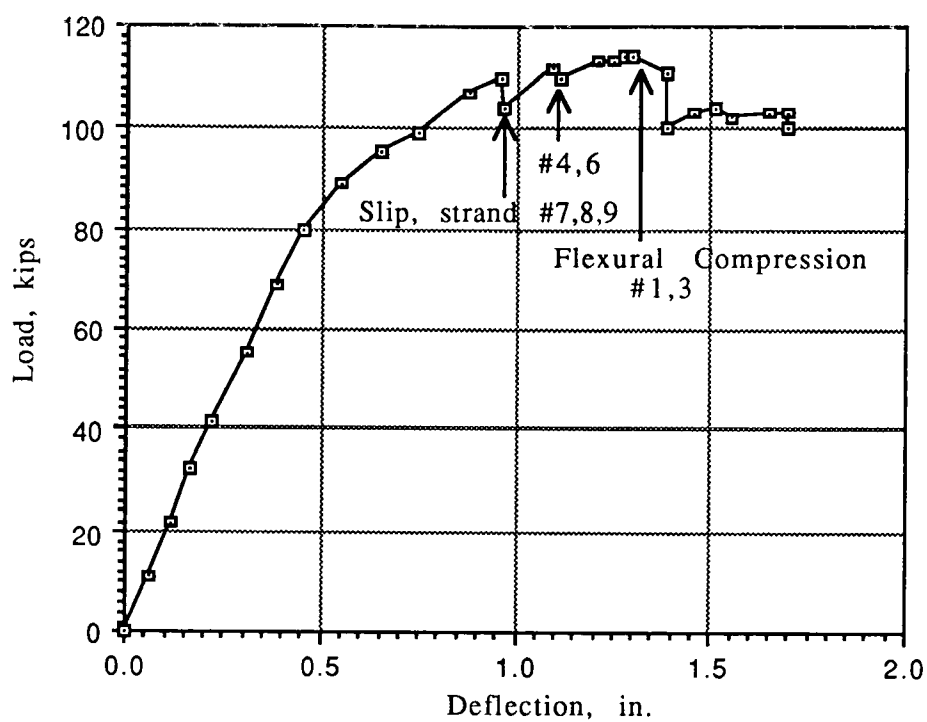


Figure 5.2 Load Vs. Deflection for Beam 916-2-EXT

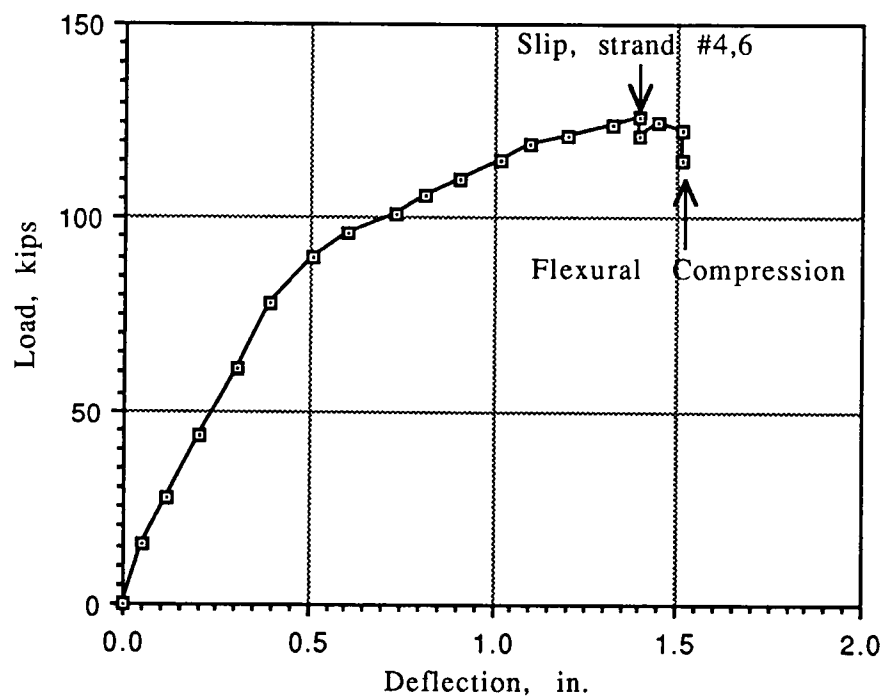
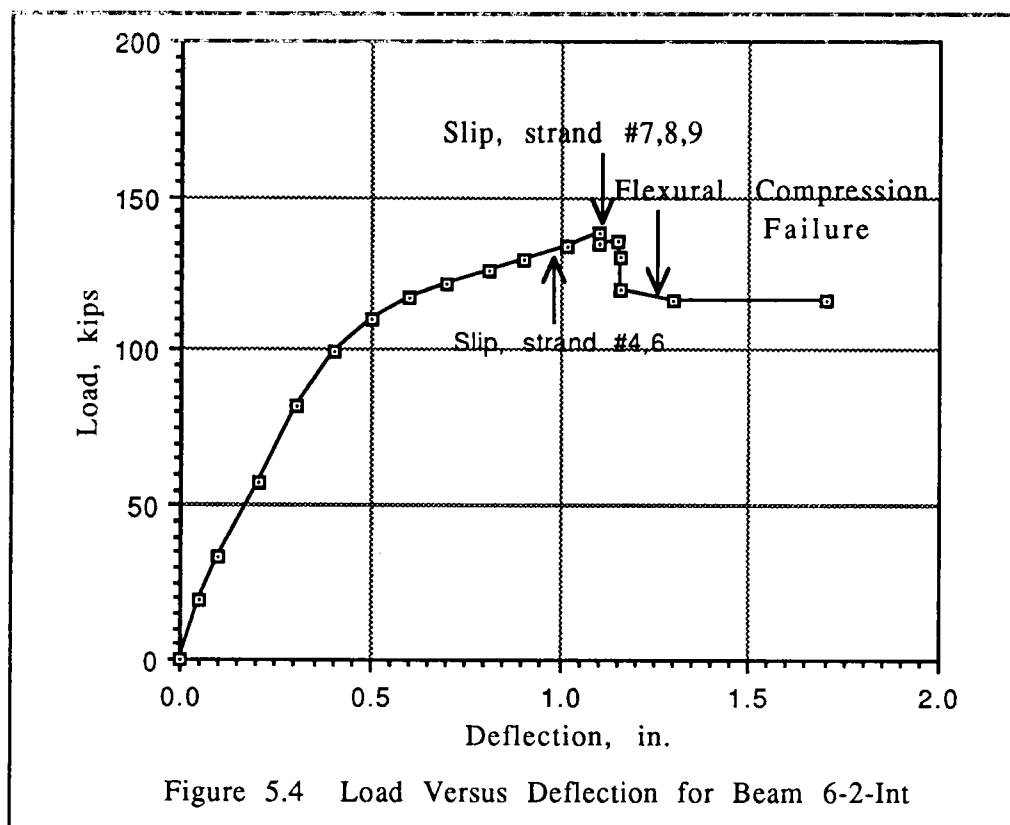


Figure 5.3 Load Versus Deflection for Beam 6-2-Ext



Increases in the steel stresses of strands that did not slip were measured by strain gages (see Figure 4.6) which confirmed that higher bond resistance was possible after slippage of some strands in the beam. As presented in Table 4.3, not all strands had slipped when the beam finally failed in flexural compression and lost most of its strength. Further increase in steel stress was also observed beyond flexural compression failure of the beam.

Some "slight" slippage of the strands during static testing did not significantly reduce a beam's flexural strength, but it did make the beam more susceptible to shear failure, especially when loaded at short shear span. Bond failure relieves the compressive strain in lower beam web to permit greater participation of the concrete to

carry tensile stresses until cracking of the concrete occurs. Shear crackings further reduce the available development length and precipitate a shear failure with a significantly reduced beam strength. Flexural compression failure of beams having web-shear cracks is often not reached. The strength of these beams has been explained previously and is further discussed in the following section.

5.3.3 Load Capacity and Mode of Failure

In accordance to the ACI equivalent concrete stress block and the strain compatibility condition with an ultimate compressive strain of 0.003 for the concrete extreme fiber, the theoretical moment strength is computed for each beam section and listed in Table 5.3 as M_n . These computed M_n values are compared to the measured M_u and M_b of the corresponding beams. As seen in the table, all beams without a bond failure had M_u exceeding the M_n . The ratios of measured to computed ultimate moments of the beams, M_u/M_n , appear to be somewhat related to the reinforcement ratios - the lower the reinforcement ratio, the higher the M_u/M_n . As explained in Appendix A3, the higher M_u/M_n ratios for the beams with lower reinforcement ratios are due to the effects of greater confinement of the compressive concrete in these beams.

All the tests with bond failure also developed M_u exceeding the M_n . Bond failure of these beams occurred also at a moment above the M_n , except four tests which developed the moments of slightly

TABLE 5.3 MEASURED MOMENT AND MODE OF FAILURE

Specimen Designation	f _c (psi)	Steel Reinforcement Ratio, (%)	Mode of Failure	Moment (kip-ft)						
				Measured			ACI	M _{cr}	M _b	M _u
				M _{cr}	M _b	M _u	M _n	M _u	M _n	M _n
5-1-EXT	5476	0.421	B-F	396	611	645	572	0.61	1.07	1.13
5-1-INT	5476	0.421	B-S	469	550	564	572	0.83	0.96	0.99
5-2-EXT	6746	0.421	B-S	444	612	612	596	0.73	1.03	1.03
5-2-INT	6746	0.421	B-F	415	623	639	596	0.65	1.05	1.07
5-3-EXT	6858	0.421	F	388	-	675	598	0.57	-	1.13
5-3-INT	6858	0.421	B-F	408	645	679	598	0.60	1.08	1.14
5-4-EXT	7600	0.421	F	410	-	676	607	0.61	-	1.11
5-4-INT	7600	0.421	B-F	453	644	664	607	0.68	1.06	1.09
5-SWAI-EAST	5553	0.417	F	430	-	641	579	0.67	-	1.11
5-SWAI-WEST	5553	0.417	F-B	485	660	660	579	0.73	1.14	1.14
5-UWR-EAST	5989	0.417	B-F	463	630	679	593	0.68	1.06	1.15
5-UWR-WEST(S)	5989	0.417	B-F	459	611	707	593	0.65	1.03	1.19
5-FWC-EAST	5341	0.417	B-F	419	624	640	572	0.65	1.09	1.12
5-FWC-WEST	5341	0.417	F	448	-	660	572	0.68	-	1.15
5-ASW-EAST	5400	0.417	F	429	-	633	574	0.68	-	1.10
5-ASW-WEST	5400	0.417	F	463	-	638	574	0.73	-	1.11
5S-1-EXT	6624	0.522	B-S	466	565	569	669	0.82	0.84	0.85
5S-1-INT(LC)	6624	0.522	F-B	449	719	724	669	0.62	1.07	1.08
5S-2-EXT	-	0.522	-	-	-	-	672	-	-	-
5S-2-INT(S)	6800	0.522	F-B	408	692	692	672	0.59	1.03	1.03
5S-3-EXT	5967	0.459	F-B	471	-	640	602	0.74	-	1.06
5S-3-INT	5967	0.459	B-S	457	587	587	602	0.78	0.98	0.98
5S-4-EXT	6181	0.459	B-S	442	530	574	606	0.77	0.87	0.95
5S-4-INT	6181	0.459	B-F	472	601	666	606	0.71	0.99	1.10
916-1-EXT	5533	0.528	B-F	488	682	688	641	0.71	1.06	1.07
916-1-INT	5533	0.528	B-F	476	633	703	641	0.68	0.99	1.10
916-2-EXT	5921	0.528	B-F	455	642	665	661	0.68	0.97	1.01
916-2-INT	5921	0.528	B-F	465	677	682	661	0.68	1.02	1.03
916-3-EXT	6119	0.384	B-F	419	628	663	557	0.63	1.13	1.19
916-3-INT	6119	0.384	F	390	-	675	557	0.58	-	1.21
916-4-EXT	6237	0.384	B-F	390	601	669	559	0.58	1.08	1.20
916-4-INT(LC)	6237	0.384	F	429	-	688	559	0.62	-	1.23
6-1-EXT	5126	0.434	F	412	-	658	595	0.63	-	1.11
6-1-INT(S)	5126	0.434	F	335	-	682	595	0.49	-	1.15
6-2-EXT	5285	0.434	B-F	398	660	660	600	0.60	1.10	1.10
6-2-INT	5285	0.434	B-F	462	609	657	600	0.70	1.02	1.10
6-3-EXT	7463	0.434	B-F	424	696	701	637	0.60	1.09	1.10
6-3-INT	7463	0.434	F	426	-	704	637	0.61	-	1.11
6-4-EXT	7984	0.434	F	406	-	695	642	0.58	-	1.08
6-4-INT	7984	0.434	F	428	-	711	642	0.60	-	1.11
5E-1-EXT	5783	0.421	F	436	-	683	579	0.64	-	1.18
5E-1-INT	5783	0.421	F	449	-	665	579	0.68	-	1.15
5E-2-EXT	6573	0.421	B-S-F	382	565	620	593	0.62	0.95	1.05
5E-2-INT	6573	0.421	F	417	-	652	593	0.64	-	1.10

NOTES: B - Bond Failure (Strands Slip), S - Shear Failure,
F-Flexural Compression Failure, (S) - Short Portion of Broken Beam,
(LC) - Loading on beam being Load Controlled

less than the M_n when bond failures occurred (i.e. tests 5S-4-INT, 916-1-INT, 916-2-EXT, and 5E-2-EXT). All the beams with bond failure were near to flexural compression failures when bond failures occurred, and the strengths of these beams did not seem to be affected significantly by the bond failures.

Only four tests did not ultimately develop the M_n . In each of these four tests, the beam experienced a shear failure soon after strand end slips were detected. One other test, i.e. test 5-2-EXT, reached a μ slightly greater than M_n when shear cracking occurs simultaneously with a bond failure, and failed in shear immediately after. Flexural compression failure of all these five beams was not reached. The beams with shear failure apparently had load capacities after shear crackings that were below the load at bond failure. These beams are previously explained to have failed due to insufficient development length (or anchorage) at the shear cracks. According to truss analogy for the type I failure as illustrated in Figure 5.1, the load capacities of these beams after shear cracking are calculated and presented in Table 5.4. The sustainable shear load after shear cracking appears to be close to the calculated tensile force that all the bottom strands in each beam can develop over the transfer bond length from the free ends of the strands to the web-shear crack that initiate from the interior face of the support. To prevent such a failure after shear cracking, a proportionate increase in support bearing distance should be provided.

Table 5.4 Strength of Beams with Bond and Shear Failures		
Specimen Designation	Shear Load after Shear Cracking V' (kips)	Tensile Force at Web-shear Crack T' (kips)
5-1-INT	74.0	76.0
5S-1-EXT	83.7	86.4
5S-3-INT	75.6	70.4
5S-4-EXT	78.0	72.8

The "minimum development lengths" of the strands to develop the computed ultimate moment, M_n , of the beams are respectively 77.4", 81", 87" and 74.4" for 1/2" Regular, 1/2" Special, 9/16" and 6/10" strands of milled condition and pretension of 202.5ksi. These minimum development lengths correspond to approximately 155 times the diameter of the strands ($155d_b$) of all the strands except the 6/10" strand. Contrary to expectation, the 6/10" strand requires only about $125d_b$ or that less than what is required for the 1/2" Regular strand. However, significantly longer development lengths are required to totally prevent any slippage of the strands at the ultimate flexural moments of the beams; Table 5.5 tabulates these lengths as the full development length. Strands exposed to weathering of up to three days in comparison to milled surface condition strands have better bonding characteristics and thus reduced development lengths. The epoxy-coated 1/2" strands performed significantly better than all the uncoated strands with a

full development length of less than 51". Reduction of strand spacing from 2" to 1.75" for 1/2" strands did not appear to cause any adverse effects on the development length. In the eight tests with the smaller spacing, there was no evidence of splitting between the strands.

Table 5.5 Minimum and Full Development Lengths		
Strand (Milled)	Minimum Development Length (in.)	Full Development Length (in.)
1/2" Regular	77.4	above 92
1/2" Special	81	above 83
9/16"	87	above 106
6/10"	74.4	86

Chapter VI

ANALYSIS OF TEST RESULTS

The UTK experimental results presented in chapter IV are influenced by several variables which are analyzed in detail in this chapter. It has been discussed in the previous chapter that the development length is composed of two effective bond length portions: one with the ultimate transfer bond, and the other with the ultimate flexural bond. The two ultimate bonds are different in magnitude, but each is practically constant over the respective bond length. In this chapter, the ultimate transfer and flexural bonds are derived from the UTK experimental data and then evaluated separately for the effects of the variables on each of the bond strengths. In addition, the correlation of the measured transfer length with the measured end slip at transfer and the correlation of the flexural bond strength with the transfer bond strength are determined. The pullout bond strength is compared to the transfer bond strength and the flexural bond strength.

6.1 Effects of Variables on Transfer Bond Strength

Although the number of tests in the UTK test program was considerable, there were also many variables affecting the bond strength. The primary variables studied for their effects on the transfer bond strength are the strand diameter and the strand

surface condition. Some unintended variations, which might affect the test results, are unavoidable in the fabrication and testing of specimens, but only the effect of the unintentional variation in concrete strength is considered and discussed here. The effects of variables like strand spacing and strand manufacturer can only be discussed for the small number of data obtained from the beam tests of a manufacturers' group. Other factors such as the type of release, degree of consolidation of the concrete around the strand, concrete mix, and concrete curing condition were held constant throughout the investigation.

6.1.1 Effects of Strand Diameter

Figure 6.1, a plot of the transfer bond strength versus the diameter of strand for all strand diameters and surface conditions, shows that the transfer bond strength is generally proportional to the strand diameter for 1/2" Regular, 1/2" Special, and 9/16" strands with a milled surface condition. The best linear fit in the figure is only for the data points of the milled surface 1/2" Regular, 1/2" Special, and the 9/16" strands. A much higher transfer bond strength is clearly observed for the 6/10" strands. This means that if the bond strength is assumed to be proportional to the diameter of strands for all the strands, a more conservative bond will be calculated for the 6/10" strand using the best-fit relationship for the smaller strands. The epoxy coated strands, although not likely to be comparable to the uncoated strands, with the diameter measured

Figure 6.1 is a scatter plot showing the relationship between Transfer bond strength (kips/in.) on the Y-axis and Diameter of strand (in.) on the X-axis. The Y-axis ranges from 0.6 to 2.0 kips/in. with major grid lines every 0.2 units. The X-axis ranges from 0.48 to 0.62 in. with major grid lines every 0.02 units. The plot includes data points for five different strand conditions and a linear regression line.

Legend:

- Milled, 2" spacing
- Weathered 3-day, 2" spacing
- × Weathered 1-day, 2" spacing
- + Weathered 3-day, 1.5" spacing
- ▲ Epoxy-coated, 2" spacing

Regression Equation:

$$U_t = -0.311 + 2.43d_b$$

$R = 0.50$

Diameter of strand (in.)	Transfer bond strength (kips/in.)	Condition
0.50	0.80	Milled, 2" spacing
0.50	0.96	Milled, 2" spacing
0.50	1.14	Weathered 1-day, 2" spacing
0.50	1.34	Weathered 1-day, 2" spacing
0.50	1.63	Weathered 3-day, 1.5" spacing
0.52	0.88	Weathered 3-day, 2" spacing
0.52	0.91	Weathered 3-day, 2" spacing
0.52	0.96	Weathered 3-day, 2" spacing
0.52	1.04	Weathered 3-day, 2" spacing
0.52	1.45	Weathered 3-day, 2" spacing
0.56	0.83	Milled, 2" spacing
0.56	0.99	Milled, 2" spacing
0.56	1.10	Milled, 2" spacing
0.56	1.22	Weathered 3-day, 2" spacing
0.56	1.36	Weathered 3-day, 2" spacing
0.56	1.60	Weathered 3-day, 2" spacing
0.58	1.33	Epoxy-coated, 2" spacing
0.58	1.47	Epoxy-coated, 2" spacing
0.58	1.55	Epoxy-coated, 2" spacing
0.58	1.85	Epoxy-coated, 2" spacing
0.60	1.35	Milled, 2" spacing
0.60	1.51	Milled, 2" spacing
0.60	1.63	Milled, 2" spacing
0.60	1.68	Milled, 2" spacing
0.60	1.85	Milled, 2" spacing
0.60	1.97	Milled, 2" spacing

Figure 6.1 shows that weathered surface conditions of strands improve the transfer bond strength. However, the improvements spread over a large range from virtually none to as much as 60%.

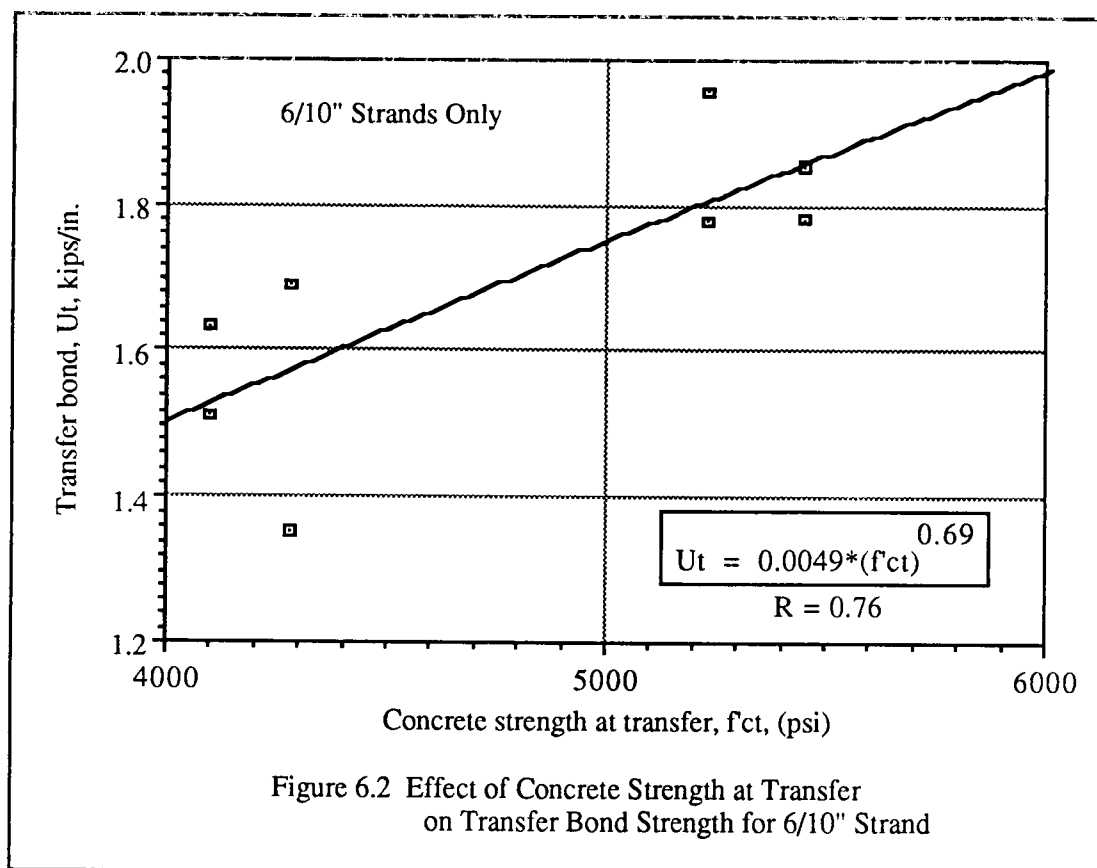
This means that the increase in bond strength can possibly provide an extra margin of safety, but is not entirely reliable. It is also noticed that there is a greater proportionate increase in the bond strength due to weathering for the 1/2" Regular strands than for the larger 1/2" Special and 9/16" strands.

The 1/2" weathered strands at the smaller strand spacing of 1.5" seem to give even better bond strength than the 1/2" strands at the 2" strand spacing. This greater increase in strength may also be caused by the longer duration of weathering in the strands at 1.5" spacing. The data for the 1.5" strand spacing spread over a range no larger than that for the other data groups. This relatively narrow spread may be evidence that there is only nominal variation in the strands from the different manufacturers.

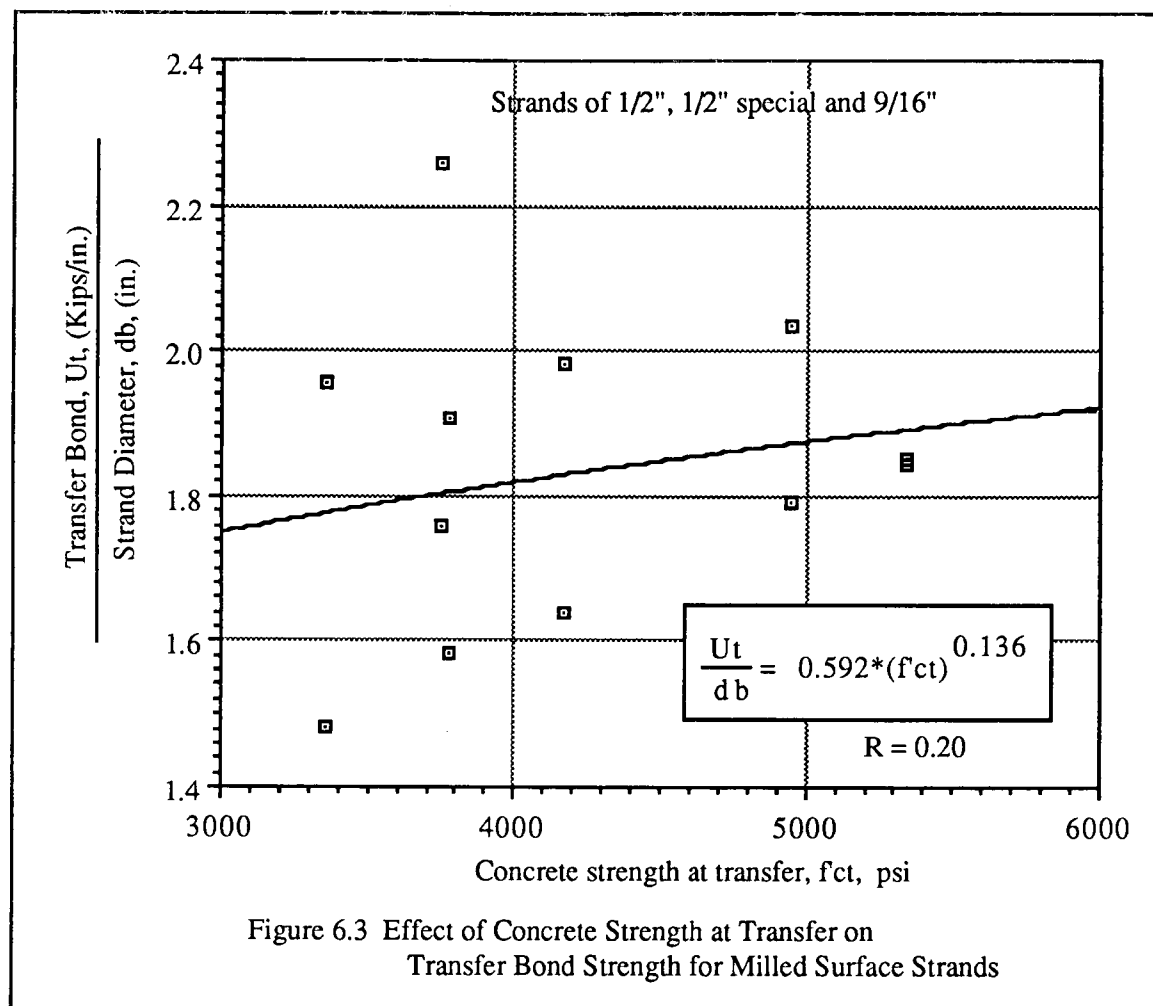
6.1.3 Effects of Concrete Strength at Transfer

Because the 6/10" strand has distinctively higher bond strength which does not follow the trend of being proportionate to the diameter of the strand, the effects of concrete compressive strength on the transfer bond strength is studied separately for 6/10" strand alone, and for the 1/2", 1/2"special, and 9/16" strands together.

The 6/10" strand group's only variation is in concrete strength. As illustrated in Figure 6.2, for the limited number of data for these strands, the transfer bond appears to be related to the concrete strength. The logarithmic fit gives the transfer bond being



proportionate to the 0.69 power of the concrete strength. However, in the following Figure 6.3, for the strands of 1/2" Regular, 1/2" Special and 9/16" diameter, concrete strength seems to have a negligible effect on the bond strength with a very low correlation. The spread in bond strength at lower concrete strengths appears larger. According to the best logarithmic fit, a variation in concrete strength from 4000 psi to 5000 psi only changes the transfer bond by a negligible 3%. Thus, the effects of concrete strength are not known conclusively.



6.2 Correlation of Transfer Length with End Slip

The end slip measured at transfer is correlated to the transfer length as in Figure 6.4. It can be seen from the figure that there is an excellent correlation between the transfer length and the end slip with a calculated correlation coefficient of 0.88. The theoretical cumulative end slip and transfer length relationship, in accordance with an assumed linear bond distribution, is calculated and compared to the measured data. The theoretical calculation of the end slip is

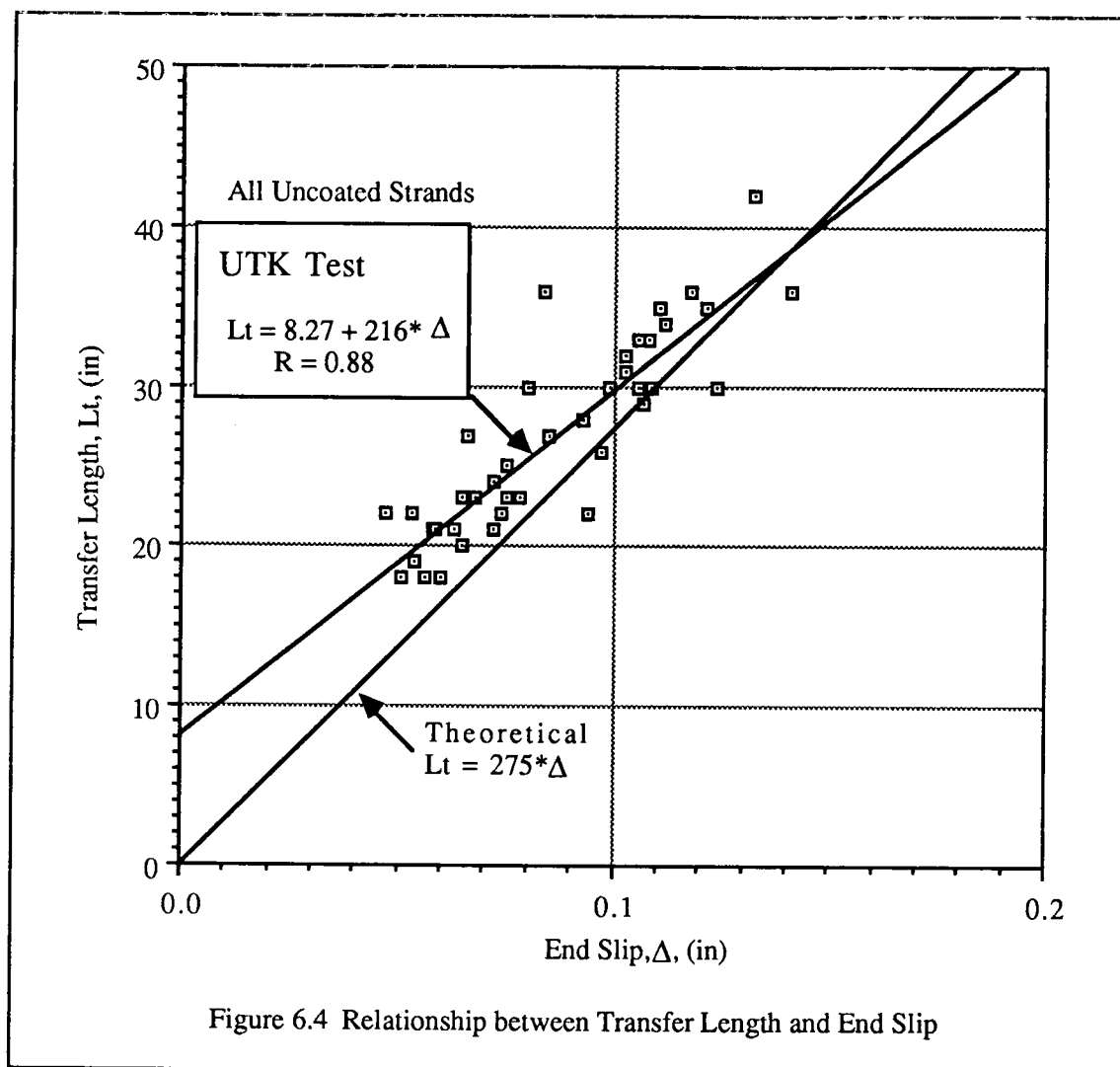


Figure 6.4 Relationship between Transfer Length and End Slip

shown in detail in Appendix A1. The closeness of the theoretical and the measured data evidently indicates that the assumed bond distribution is valid and that the actual transfer length may be quite closely predicted using a calibrated theoretical relationship for any measured end slip.

6.3 Flexural Bond from Load Tests

The average ultimate flexural bond (or the flexural bond strength) can be deduced from the load tests of beams which exhibited a bond failure. The steel stress developed at the critical section, which is approximately at the load point, must be known in order to determine the flexural bond strength. Table 6.1 summarizes the steel stresses that were measured by the electrical strain gages, f_{sg} , and those which are calculated, f_{sc} , from the beam strength measured in the development length tests. The steel stress developed at the critical section of a loaded beam is calculated from the measured ultimate moment; the details of calculations for the steel stress are explained in Appendix A3. The calculated steel stresses generally compare quite well with those measured by the available electrical strain gages on the strands at the load points. When strain gages are available at the load points, the steel stresses measured by the gages are selected over the theoretical values for use in the flexural bond strength analysis. The theoretically calculated steel stresses are substituted only for the test beams which did not have a functional strain gage at the load point.

The flexural bond length is the length from the end of the transfer bond zone to the critical section. For a beam that had electrical strain gages within the flexural bond length, an ineffective bond length of 3", taken up by the strain gages, is subtracted from the flexural bond length. The additional steel stress built up over the

TABLE 6.1 SUMMARY OF FLEXURAL BOND RESULTS

SPECIMEN DESIGNATION	f _c (psi)	Transfer		Effective Load Location	Gage Location	Steel Stress@ Measured		Mode of Failure	fsu - fse (ksi)	Flexural Bond Length Lb (in.)	Average Flexural Bond	
		f _c (psi)	Length Lt (in.)	f _s (ksi)	Ld (in.)	f _{se} (ksi)	fsu (ksi)				(ksi/in.)	(kips/in.)
5-1-EXT	5476	36	186	-	92	265	270	B-F	79	56.00	1.41	0.215
5-1-INT	5476	30	187	-	69.6	240	232	B-S	53	39.60	1.34	0.205
5-2-EXT	6746	35	188	-	77.4	259	248	B-S	71	42.40	1.68	0.257
5-2-INT	6746	29	188	-	85	242	260	B-F	72	53.00	1.37	0.209
5-3-EXT	6858	23	190	68.00	93	-	276	F	87	62.10	1.40	0.214
5-3-INT	6858	22	191	72.00	81	265	278	B-F	88	55.40	1.59	0.243
5-4-EXT	7600	22	192	77.00	81.25	260	274	F	83	56.25	1.47	0.225
5-4-INT	7600	26	192	65.40	77.4	-	268	B-F	77	51.40	1.51	0.230
5-SWAI-EAST	5553	20	193	-	81	269	265	F	76	61.00	1.25	0.191
5-SWAI-WEST	5553	21	193	-	69.6	270	274	F-B	81	48.60	1.68	0.256
5-UWR-EAST	5989	21	194	-	77.4	-	280	B-F	86	56.40	1.53	0.235
5-UWR-WEST(S)	5989	19	194	-	69.6	-	286	B-F	92	50.60	1.82	0.279
5-FWC-EAST	5341	18	192	-	72.5	-	266	B-F	75	54.50	1.38	0.211
5-FWC-WEST	5341	18	191	-	77.4	-	275	F	85	59.40	1.43	0.219
5-ASW-EAST	5400	21	193	-	73.5	-	262	F	70	52.50	1.34	0.205
5-ASW-WEST	5400	18	192	-	69.6	-	265	F	73	51.60	1.42	0.217
5S-1-EXT	6624	33	190	-	69	232	196	B-S	42	36.00	1.16	0.193
5S-1-INT(LC)	6624	33	191	-	81	217	263	F-B	72	45.00	1.60	0.268
5S-2-EXT	-	34	191	-	84	-	-	-	-	-	-	-
5S-2-INT(S)	6800	30	191	-	82.5	260	230	F-B	69	52.50	1.31	0.219
5S-3-EXT	5967	31	191	-	81	253	256	F	66	47.00	1.41	0.235
5S-3-INT	5967	36	190	-	75	245	230	B-S	55	39.00	1.41	0.236
5S-4-EXT	6181	35	190	44.00	68	243	223	B-S	53	33.00	1.61	0.268
5S-4-INT	6181	22	190	67.00	72	255	267	B-F	78	50.00	1.55	0.259
916-1-EXT	5533	42	183	90.00	106	240	254	B-F	65	64.00	1.02	0.196
916-1-INT	5533	32	184	74.00	87	242	263	B-F	74	55.00	1.35	0.259
916-2-EXT	5921	36	186	77.00	96	-	236	B-F	51	57.00	0.90	0.172
916-2-INT	5921	28	186	80.00	87	-	244	B-F	59	59.00	1.00	0.192
916-3-EXT	6119	30	191	-	95.5	114	271	B-F	80	65.50	1.22	0.235
916-3-INT	6119	23	191	-	104.4	105	276	F	85	81.40	1.05	0.201
916-4-EXT	6237	30	191	88.40	104.4	246	271	B-F	81	71.40	1.14	0.218
916-4-INT(LC)	6237	27	191	87.00	108	251	281	F	90	78.00	1.16	0.222
6-1-EXT	5126	25	190	-	116	116	261	F	72	91.00	0.79	0.172
6-1-INT(S)	5126	27	190	-	93	66	263	F	73	63.00	1.16	0.252
6-2-EXT	5285	30	189	75.00	83.5	244	262	B-F	73	53.50	1.37	0.298
6-2-INT	5285	24	189	64.00	74.4	92	260	B-F	72	50.40	1.43	0.310
6-3-EXT	7463	23	191	71.50	83.52	85.5	266	B-F	76	60.52	1.25	0.272
6-3-INT	7463	21	191	76.20	88.16	93	267	F	77	67.16	1.14	0.248
6-4-EXT	7984	22	190	73.80	85.84	89	262	F	72	63.84	1.13	0.245
6-4-INT	7984	23	191	62.80	85.84	95.5	268	F	78	62.84	1.24	0.269
5E-1-EXT	5783	20	193	47.00	58.08	78.00	287	F	95	38.08	2.50	0.383
5E-1-INT	5783	19	193	48.70	69.66	55.00	278	F	85	50.66	1.68	0.258
5E-2-EXT	6573	22	192	30.00	46.40	65.00	253	B-S-F	77	24.40	3.15	0.482
5E-2-INT	6573	16	193	41.00	51.10	69.50	268	F	76	35.10	2.15	0.329

flexural bond length is calculated by subtracting the effective prestress at transfer from the steel stress developed at the critical section. (The maximum steel stress that may develop over the transfer length is reasonably assumed to be approximately equal to the effective prestress. Transfer bond is plastic over virtually the entire transfer bond length and is therefore assumed to be incapable of sustaining any additional bond.) The additional steel stress multiplied by the area of the strand gives the additional force developed in the strand over the flexural bond length. The average flexural bond is the force divided by the flexural bond length. These calculated values of average flexural bond for all the beams are presented in Table 6.1.

It must be emphasized that only the beams for which strand slippage is detected are considered to have reached the bond strength over the entire development length of the strands; these beams are designated with B-F or B-S for the failure modes. Strand slippage is an indication that the limit state for a development length has been reached; this is the state in which the flexural bond first becomes all plastic, and all elastic adhesion bond over the development length is broken. The beams which develop flexural compression failure without any strand slippage (F) are considered to have a portion of the flexural bond length still in the elastic range and not fully effective. For these "F" beams, the average flexural bond calculated is below the flexural bond strength.

All the beams tested were observed to have flexural cracks and/or shear cracks closer to the ends of the beams, away from the load points. Photographs taken of beams with bond failure revealed that when the strands in each beam slipped, the flexural crack closest to the end of the beam propagated toward the load point and upward into the flexural compression zone, and finally caused a flexural compression failure to occur above the tip of the crack. The flexural crack is similar to what is generally referred to as "flexural-shear crack." Figure 6.5 and Figure 6.6 show the photographs of two typical beam tests that failed in this manner: 916-1-EXT and 6-2-INT. The critical sections for the beams appear to be formed by the flexural-shear crack. The longitudinal distance from the load point to the point of crossing of the flexural-shear crack with the centroidal axis of bottom steel ranges from 5" to approximately the depth of the bottom steel. This short bond length within the flexural bond length is considered to be relatively ineffective in bond resistance. Thus, the effective development length for each of these beams is from the crack to the end of the beam. Accordingly, the flexural bond strength should be deduced from the steel stress developed at this critical section instead of that at the load point. However, the information on the location of the flexural-shear crack is not available for all the beam tests. For each of the beams with such information available, Table 6.1 tabulates the length from the flexural shear crack to the end of the beam, hereby called "effective L_d ." Most unfortunately, this information is not available for the



Figure 6.5 Photograph of Flexural-Shear Cracks in Beam 916-1-EXT



Figure 6.6 Critical Section at Flexural-Shear Crack: Beam 6-2-INT

1/2" Regular and 1/2" Special strands with milled surface condition. In view of this limitation, the analysis of flexural bond strength for all the strands is based on the flexural bond strength that is deduced from the longer development length which is from the assumed critical section at the load point to the end of the beam; the flexural bond strength obtained in this manner would be lower and more conservative. Although the flexural bond strength may be slightly conservative for shallower beams, the similar cracking conditions probably exist in deeper beams, and the average bond strength as derived would be reasonable. Beams deeper than the UTK bridge beams would probably span farther in practice; and development length in longer beams is unlikely to be critical. Thus, the flexural bond strength obtained is likely to be conservative.

Flexural compression failure did not occur in beams with bond failures and which suffered web-shear cracking and immediately lost significant portions of their load capacities, which indicates that the maximum load capacities were not limited by the flexural strengths. The steel stresses calculated in the same manner as that for "B-F" beams are not valid. Fortunately for all these "B-S" beams, electrical strain gages were available at the load points to monitor the steel stresses. Before shear cracking, these beams only had the first flexural cracks at the load points; therefore, the available development length is the length from the load point to the end of each beam. If bond strength is fully developed over the entire development length, the calculated flexural bond strengths would be

higher than those conservatively calculated for the beams with "B-F" failures. But the fact that the calculated flexural bond strengths for these beams are not apparently higher than those for "B-F" failures indicates that the flexural bond strength may not have fully developed over the entire flexural bond length when shear failure occurred. These beams were also examined in the previous chapter for the influence of the shear cracks on development length and beam behavior.

6.4 Effects of Variables on Flexural Bond Strength

The calculated average flexural bond strength is analyzed here for the effects of the variables affecting bond strength similar to the analysis for the transfer bond strength in Section 6.1. The following variables are separately evaluated for their effects: 1) the strand diameter, 2) the strand surface condition, and 3) the concrete strength at load test. The effects of strand spacing and strand manufacturers are briefly discussed.

6.4.1 Effects of Strand Diameter

Figure 6.7 shows a plot of average flexural bond strength versus strand diameter for beams with strands of milled surface condition that experienced a bond failure, "B-F" beams. Also shown in the figure are the average flexural bond strengths developed for beams with milled strands that did not have bond failure, "F" beams. The flexural bond strength developed in the "F" beams is clearly

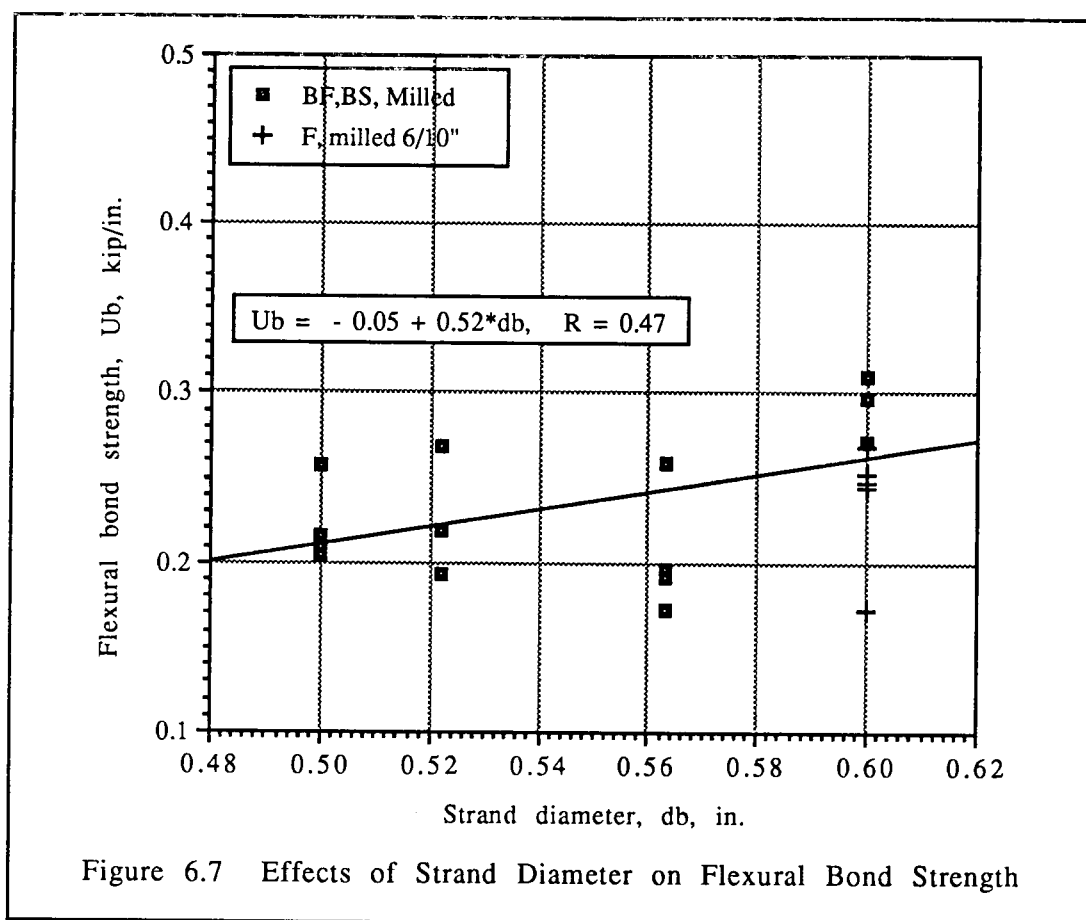


Figure 6.7 Effects of Strand Diameter on Flexural Bond Strength

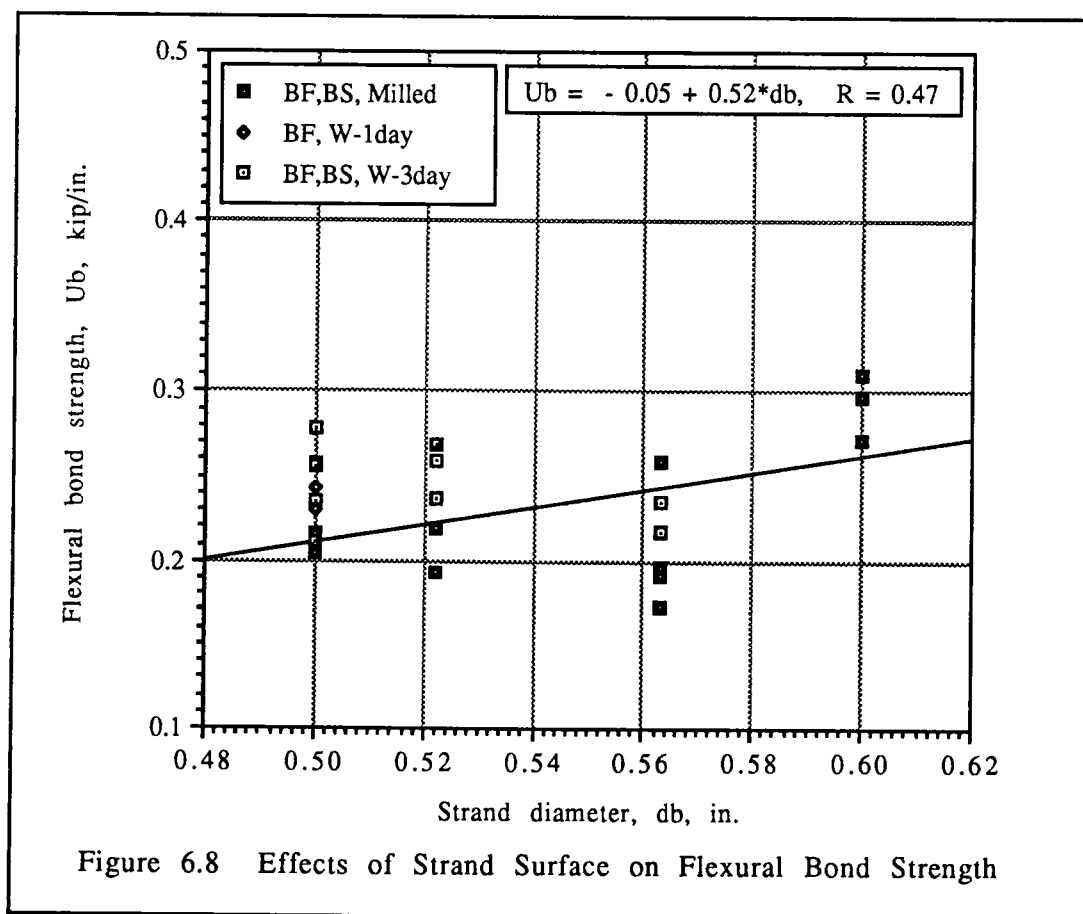
lower than in the "B-F" beams. The reason for the lower strength is that the flexural bond strength was not fully developed over the entire available flexural bond length of the "F" beams. Thus, only the data for the "B-F" beams are studied for the effects on the bond strength.

The flexural bond strength appears to be related to strand diameters for all strands. The best linear regression line for all the milled "B-F" data fits all the strand diameters except the 9/16" strands. The 9/16" strands have a relatively lower flexural bond strength, but deviation from the trend for the other strands is not obvious. The flexural bond strength in kips/in. for all the milled

strands is approximately equal to 0.52 times the strand diameter in inches.

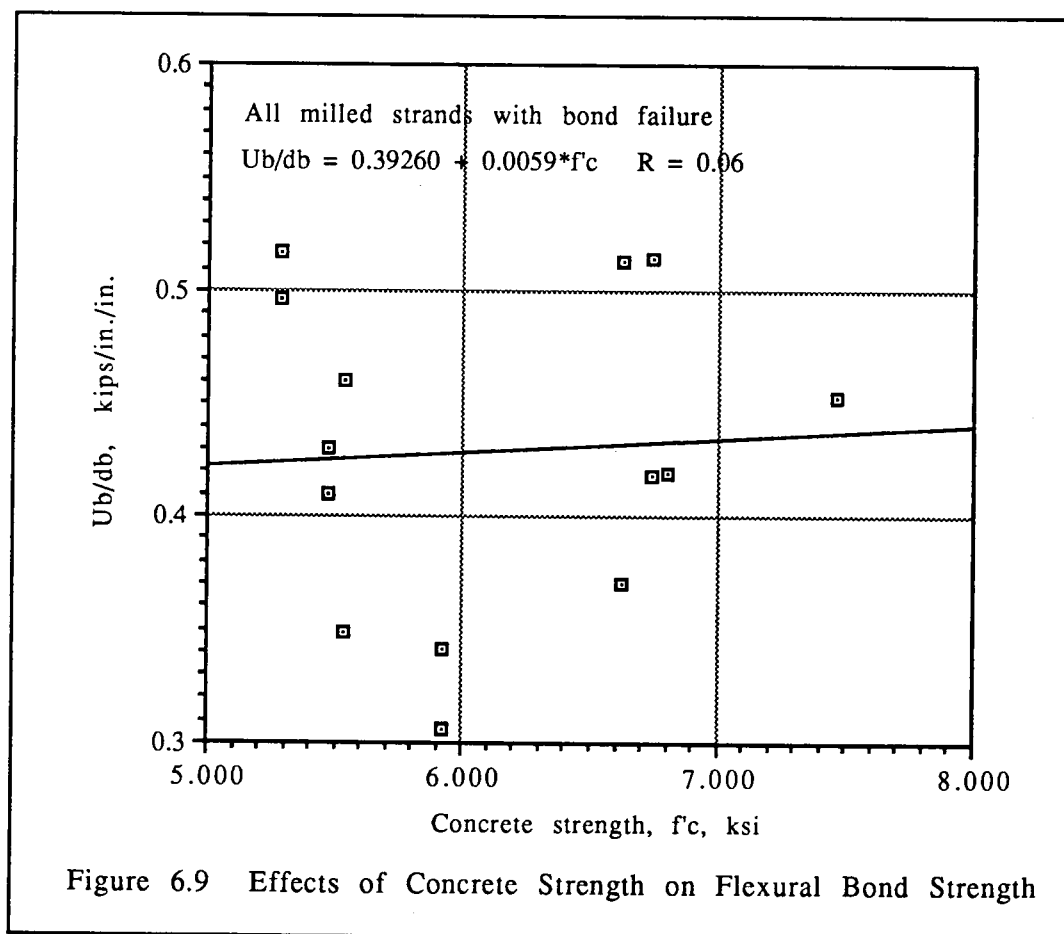
6.4.2 Effects of Strand Surface Condition

Figure 6.8 shows that some small improvement in the flexural bond strength may be expected for weathered strand surface condition although the reliability of such improvement seems to be low. The smaller 1.75" spacing of strands from the different manufacturers appears not to have any adverse effects on the flexural bond strength.



6.4.3 Effects of Concrete Strength

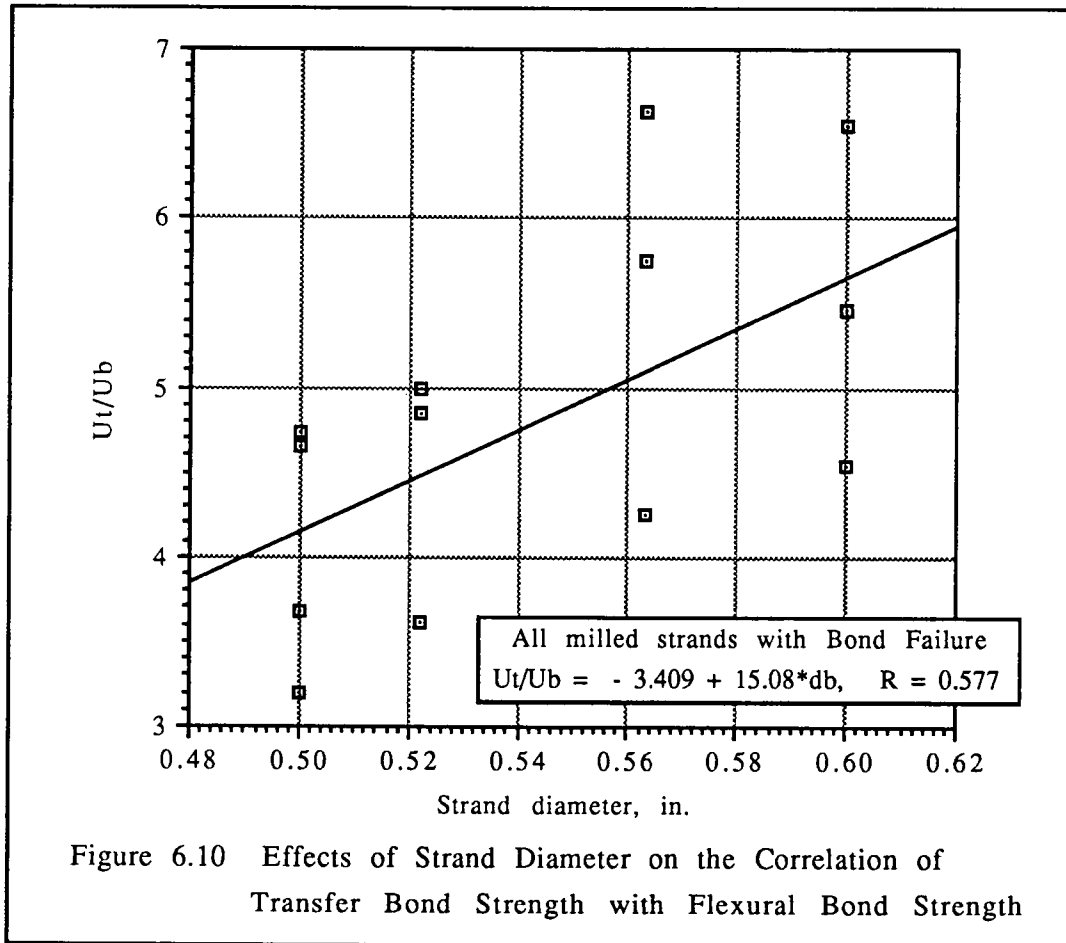
Figure 6.9 shows for all the milled surface strands with bond failure that the effects of concrete strength from 5000 psi to 7500 psi on flexural bond strength are negligible without correlation.



6.5 Correlation of Flexural Bond with Transfer Bond

Brooks et. al. [3] assumed that transfer bond strength gave a direct indication of the actual flexural bond strength. The equations for development length by H&K and Cousins imply perfect

correlation as each of the two equations has the transfer bond and flexural bond defined in terms of the same variables. To check the variation of flexural bond with the corresponding transfer bond, the corresponding bond strengths are examined for any such correlation. It can be seen from Figure 6.10 that the transfer bond strength is not



only correlated to the flexural bond, but the correlation is also related to strand diameter. The best fit line indicates that the transfer bond strength is related to the flexural bond strength with a ratio that varies from about 4.2 for 1/2" strands to 5.7 for 6/10"

strands. The figure also shows a significant scatter in the ratio for each strand diameter. For the 1/2" strands, the ratio ranges from a minimum of 3.8 to a maximum of 4.8. These ratios are close to that for H&K and Cousins which are 3 and 5.1, respectively.

The increase in the ratio with strand diameter implies that the increase in bond strength with strand diameter is faster for transfer bond than for flexural bond, which is consistent with the knowledge that mechanical bond is more significant in the transfer length due to greater relative slip and also stronger for larger strands due to the larger grooves between the wires of larger strands.

6.6 Effects of Variables on Pullout Bond Strength

Although the number of pullout prisms tested was small, the effects of the strand diameter and the strand surface condition on the average pullout bond strength are apparent. and the relationships are consistent with that found for the transfer bond strength. Figure 6.11 shows, despite considerable spread in the bond strength, that the pullout bond strength is proportional to the strand diameter for the milled strands of 1/2" Regular, 1/2" Special and 9/16" in diameter. The best linear fit in the figure is for the 1/2" Regular, 1/2" Special and 9/16" strands with a milled surface condition. The milled surface 6/10" strands clearly have much higher bond strength than would be predicted with the trend of linear proportionality with strand diameter for the smaller strands.

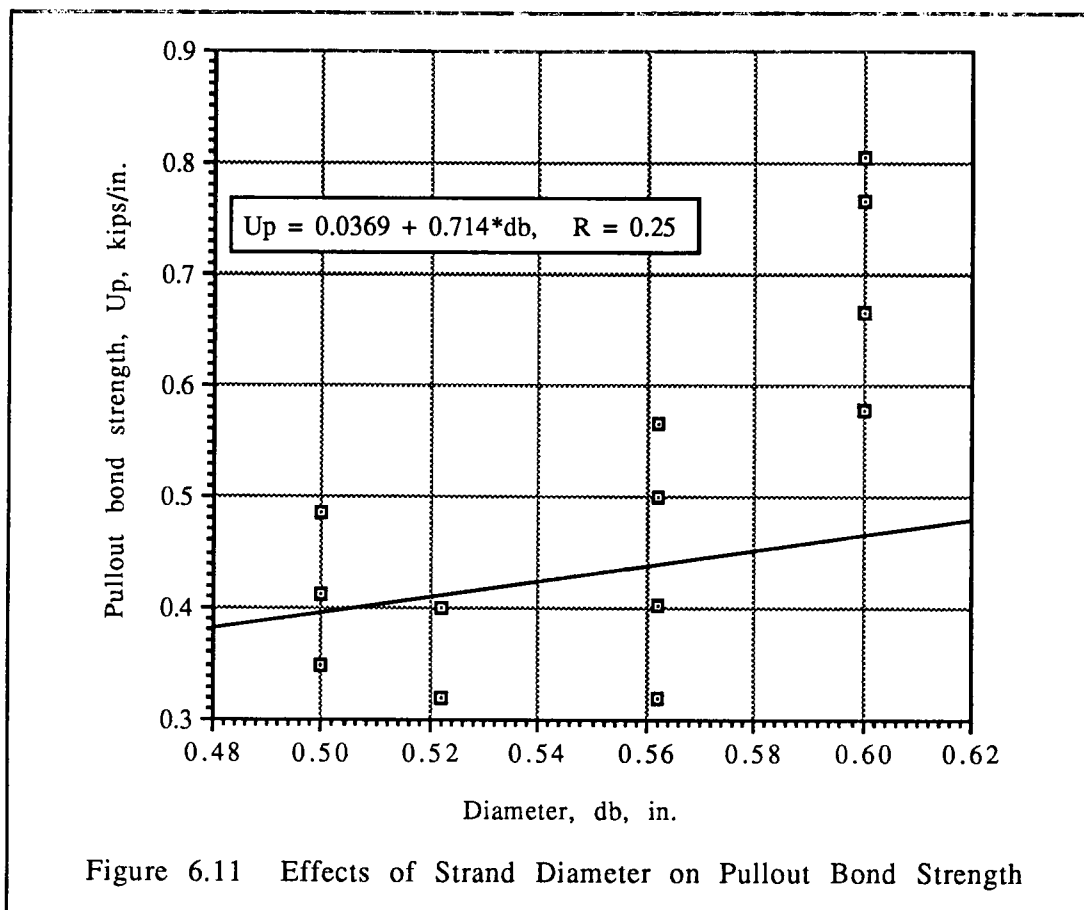
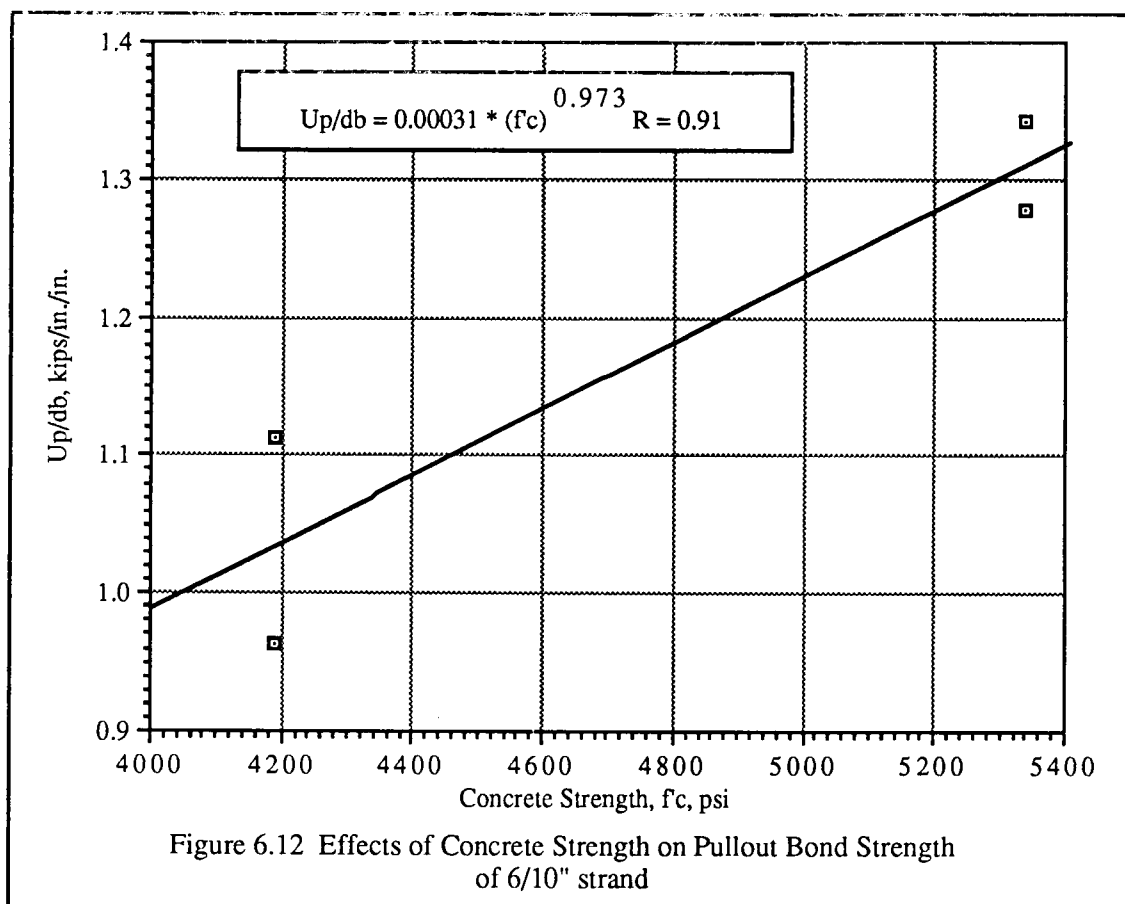


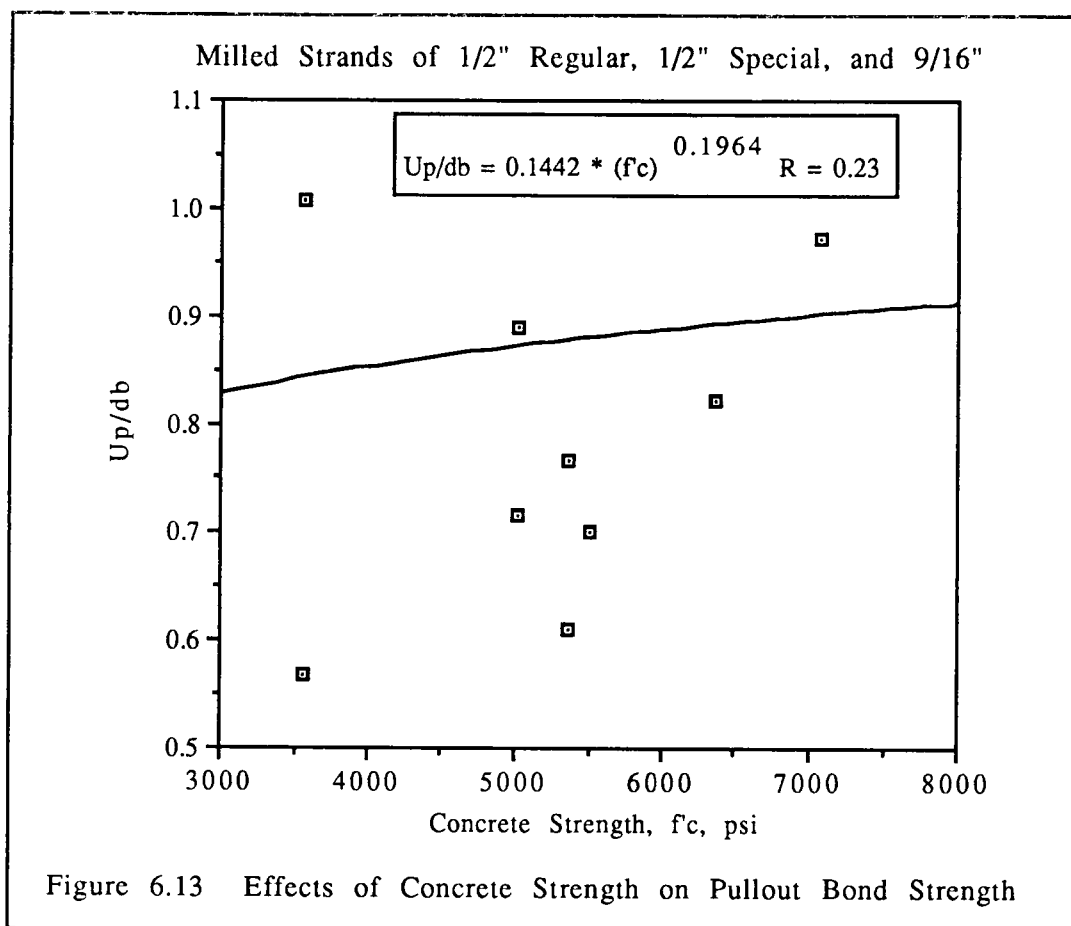
Figure 6.11 Effects of Strand Diameter on Pullout Bond Strength

The embedment lengths of 18" and 36" did not seem to affect the average pullout bond strength; the ratios of the bond strengths for the two prisms from each beam cast of 18" and 36" in embedment lengths, has an average of 1.00 but spread considerably from 85% to 178%. The fact that the average bond strength is independent of embedment length supports the theory of uniform plastic bond strength along a strand. A weathered surface of 3-day's duration significantly improves the bond strength of the strands, ranging from 180% to 240% that of milled surface.

For the 6/10" diameter strand in which the only variable was concrete strength, Figure 6.12 shows the best logarithmic fit for the



small number of data, which indicates that the pull-out bond strength is proportional to the 0.973 power of the concrete strength. However, for the 1/2", 1/2" special and 9/16" strands, Figure 6.13 shows that the pull-out bond strengths are affected much by the concrete strength and with a low correlation. Thus, the effects of concrete strength are not known conclusively.



6.7 Transfer, Pullout, and Flexural Bond Strengths

From the analysis of the effects of the variables on the pullout, transfer, and flexural bond strengths, the quality and the quantity of each type of bond is compared. Transfer and pullout bond strengths are affected quite similarly by strand diameters, concrete strength, and strand surface conditions. Flexural bond is affected by the strand diameter but not so significantly, and surface condition has less effect on flexural bond than on transfer and pullout bond strengths. Transfer bond has the greatest strength because friction bond is the highest. Pullout bond experiences relatively large slip,

therefore sustaining a significant amount of mechanical bond. Flexural bond is of the least magnitude, possibly due to a reduction of friction force from the draw-out of strand and without sufficient strand slippage to develop additional mechanical bond.

Chapter VII

RECOMMENDATIONS

The UTK test data, supported by data from other testing, are used to develop design recommendations. Comparisons of existing equations are made first; then any necessary adjustment to improve the prediction is evaluated. It was shown previously that the transfer length correlates with the measured end slip and that the flexural bond strength correlates with transfer bond strength; thus the prediction of development length may be improved by correlation with the measured end slips of the strands at transfer.

7.1 Comparison of Existing Prediction Equations.

The many proposed prediction equations for the development length of seven-wire prestressing strands at first look confusing. In a detailed comparison, it is clear that some of the equations imply that bond strength is dependent only on the diameter of strand, while others imply that the bond strength is also dependent on the concrete strength. The prediction equations for development length include those adopted by ACI and AASHTO, which evolved mainly from the research by Hanson and Kaar (referred to herein as H&K) [15], and those formulated by Cousins, et. al. (Cousins) [6], Zia and Mostafa (Z&M) [39], and Martin and Scott (M&S) [25]. The equations

by Z&M and M&S are from a reevaluation of earlier experimental data. These equations are summarized below:

H&K $L_t = (f_{se}/3)d_b$

$$L_b = (f_{ps} - f_{se})d_b$$

Cousins $L_t = (.5U'_t\sqrt{f'_c})/B + f_{se}A_s/(\pi d_b U'_t\sqrt{f'_c})$

$$L_b = (f_{ps} - f_{se})(A_s/(\pi d_b U'_d\sqrt{f'_c}))$$

where for uncoated strands:

$$U'_t = 6.7, B = 300 \text{ psi/in, and } U'_d = 1.32$$

Z&M $L_t = 1.5(f_{si}/f_{ci})d_b - 4.6$

$$L_b = 1.25(f_{su} - f_{se})d_b$$

M&S $f_{ps} = L_x[135/(d_b^{1/6}) + 31]/80d_b, \quad \text{for } L_x \leq 80d_b$

$$f_{ps} = 135/(d_b^{1/6}) + (.39L_x/d_b), \quad \text{for } L_x \geq 80d_b$$

where: A_s - area of strand (in²),
 d_b - diameter of strand (in),
 f_{ci} - compressive strength of concrete at transfer (ksi),
 f'_c - compressive strength of concrete in service (psi),
 f_{ps} - stress in prestressed reinforcement at nominal strength (ksi),
 f_{se} - effective prestress in strand (ksi),
 f_{si} - initial prestress in strand (ksi),
 f_{su} - ultimate stress of prestressing steel (ksi),
 L_b - the flexural bond length of strand (in),
 L_t - transfer length of strand (in),
 L_x - distance from end of beam to a section (in).

In the equations by H&K, Cousins, and Z&M, the development length is the total of the transfer length and the flexural bond length, i.e. $L_d = L_t + L_b$. On the other hand, M&S's equation predicts the steel stress that can be achieved for an available development length.

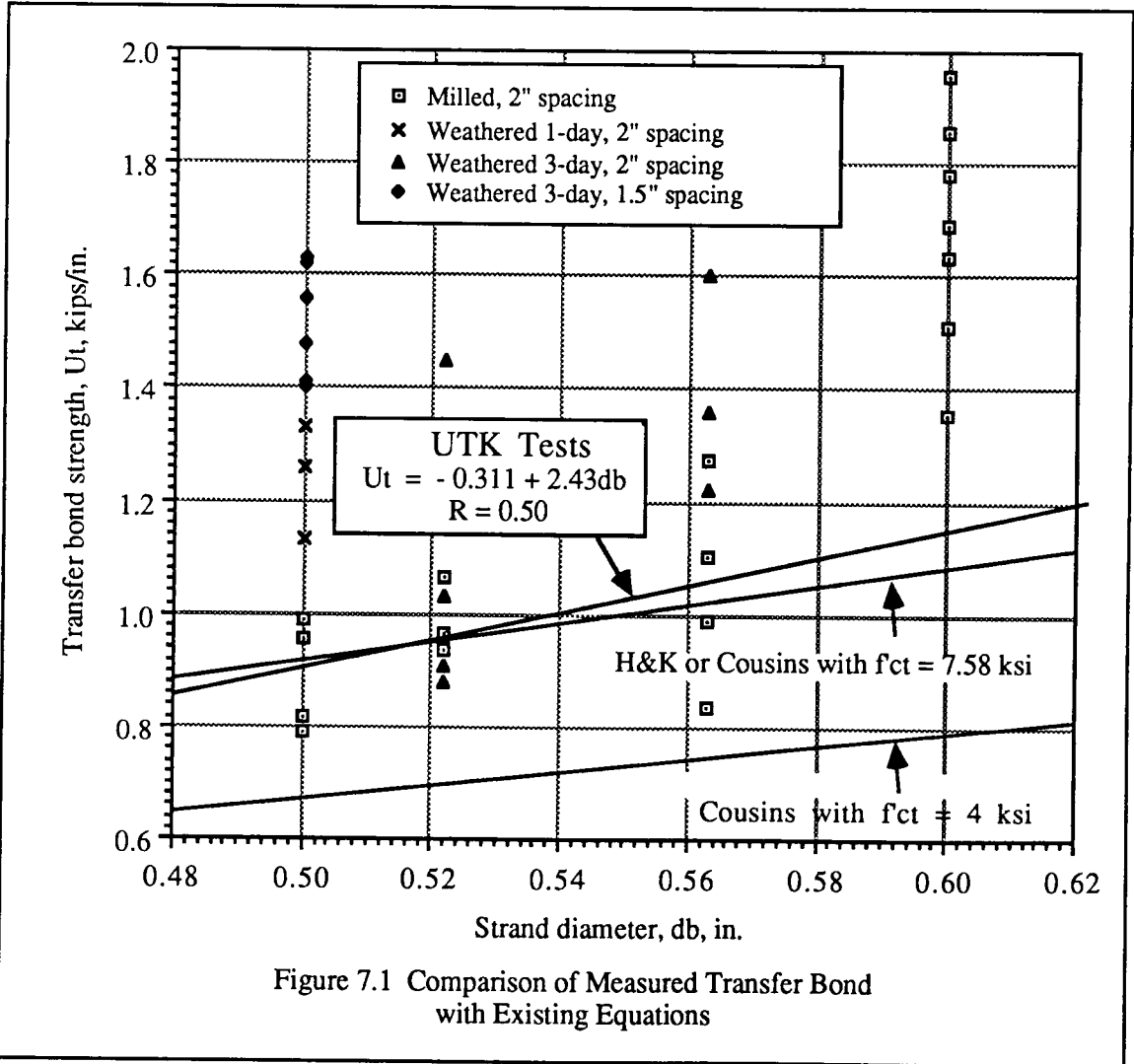
With the knowledge that the area of steel is a function of the strand diameter as explained in Appendix A2, all the equations can be rewritten into virtually the same basic form which shows the term involving average bond strength in the unit of kips per inch of bond length. For a concrete strength of 4000 psi and with the relationship of $A_s = 0.608d_b^2$, the prediction equations are rewritten to show the implied transfer bond strength, U_t , and flexural bond strength, U_b .

H&K	$U_t = (A_s f_{se} / L_t) \cong 1.82d_b$ $U_b = A_s(f_{ps} - f_{se}) / L_b \cong 0.608d_b$
M&S	$U_t = A_s f_{ps} / L_x \cong 0.0076d_b[135/(d_b^{1/6}) + 31]$, for $L_x \leq 80d_b$ $\cong 1.53d_b$ to $1.39d_b$, for 1/4" to 1/2" strand, respectively. $U_b = A_s[f_{ps} - 135/(d_b^{1/6})] / L_x \cong 0.237d_b$, for $L_x \geq 80d_b$
Z&M	$U_t = A_s f_{se} / (L_t + 4.6) \cong 0.405(f_{se}/f_{si})d_b f_{ci} \cong 1.35d_b$ for a 20% loss from f_{si} to f_{se} , and $f_{ci} = 4$ ksi $U_b = A_s(f_{ps} - f_{se}) / L_b \cong 0.486d_b$
Cousins	$U_t = (A_s f_{se} / L_t) \cong (\pi d_b U'_t \sqrt{f'_c}) \cong 21.0d_b \sqrt{f'_c} \cong 1.33d_b$ $U_b = A_s(f_{ps} - f_{se}) / L_b \cong 4.14d_b \sqrt{f'_c} \cong 0.262d_b$ neglecting the "elastic" bond term, $(.5U'_t \sqrt{f'_c})/B$, which represents a constant transfer length of merely 0.71".

In the new forms, clearly H&K's equation predicts the highest transfer and flexural bond strengths while Cousins's predicts the lowest bond strengths for the concrete strength considered.

7.1.1 Comparison of Transfer Bond Strength Equations.

A comparison of the transfer bond strength implied in the equations of H&K and Cousins with the transfer data from the UTK test program is presented in Figure 7.1. The mean for the UTK

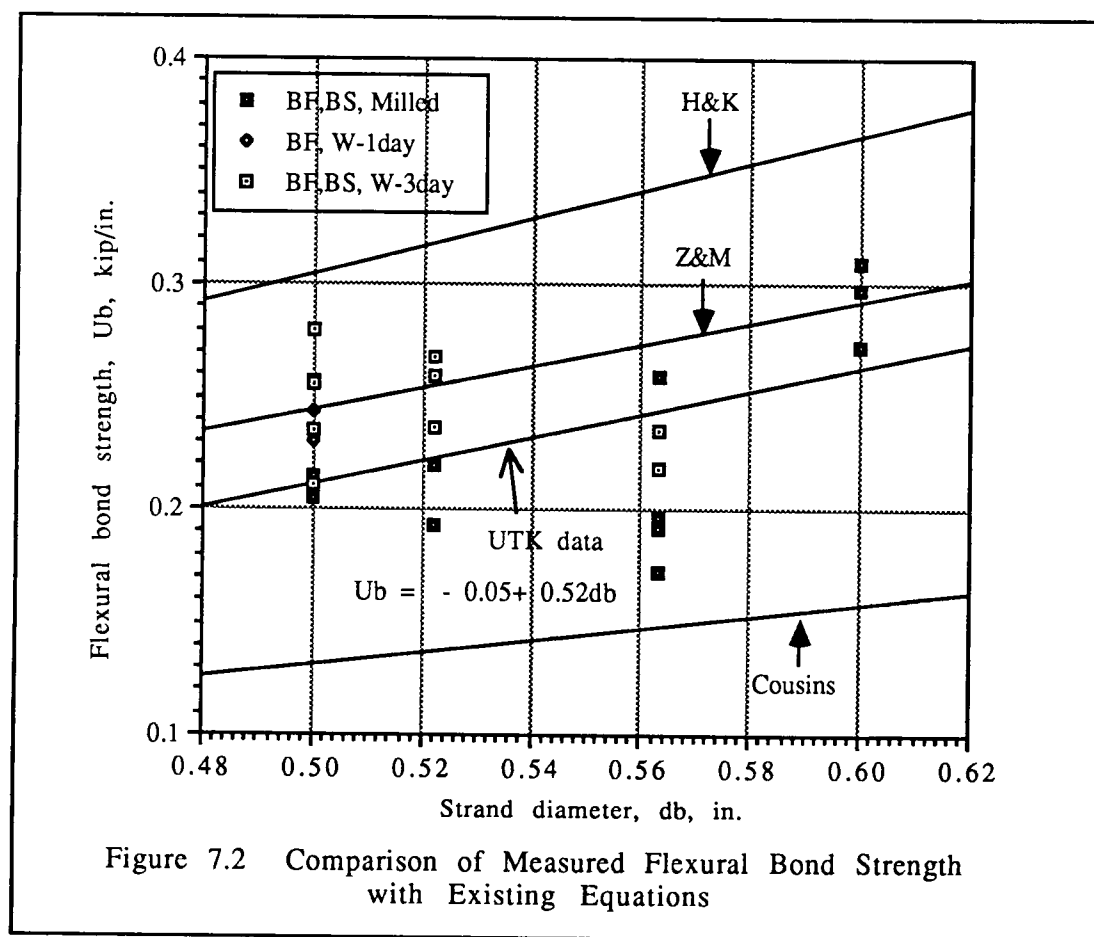


transfer bond strength with a milled surface condition, except that of 6/10" strand which has higher bond strength, corresponds well to that predicted by the equation of H&K. It needs to be pointed out that there is no difference in the two equations by Cousins and H&K except that the transfer bond strength is related to the concrete strength in the equation by Cousins. The two equations predict the same bond strength when the concrete strength at transfer is at 7580psi, but the equation by Cousins predicts a more conservative bond strength at lower concrete strengths. At the 4000psi concrete strength often encountered in practice, the equation by Cousins is about 38% more conservative than that by H&K. The concrete strength at transfer in the UTK test program ranged from 3360 to 5300 psi. As discussed previously, there is no systematic variation of the bond strength with the concrete strength; thus, the equation by Cousins appears to be too conservative to match the UTK experimental data.

7.1.2 Comparison of Flexural Bond Strength Equations.

Figure 7.2 shows a comparison of the flexural bond strength derived in the UTK beam tests with the flexural bond strengths implied in the existing equations. The flexural bond strength implied in the ACI code equation (or H&K) and the other equations by Z&M and Cousins differ greatly. Although with significant scatter, the UTK test data fall within the upper bound and lower bound established by H&K's and Cousins' equations. The H&K equation appears to be

unconservative for the UTK test data. However, it may be recalled that the flexural bond strength derived from the UTK development length tests is with the more conservative longer development length from the assumed critical section at the load point, instead of the actual development length where the critical section was actually observed to be. Thus, the Cousins' equation again would seem to be too conservative for the UTK test data.



7.2 Recommended Development Length Equation

The UTK test results appear within the range predicted by the existing development length equations. This indicates the relevancy of past research to what may actually be expected in a typical producer environment similar to that in the UTK full scale beam test program. Because development length is affected by many factors, and different methods were used to interpret test data, the test results presented in past research varied considerably. Therefore, only the equations developed by the previous investigators are examined for their applicability for all the UTK strands.

H&K's research was with smaller strands of up to 1/2" in diameter, but their equation appears to predict transfer bond strength reasonably well up to 9/16" strand. However, as the UTK test data indicate, the flexural bond strength predicted by H&K's equation seems to be unconservative.

The UTK test program includes the larger 1/2" Special, 9/16", and 6/10" strands. Only Kaar, LaFraugh, and Mass [20] and Cousins et. al [6] did experiments with strands larger than 1/2" Regular. The only larger strand that Kaar, LaFraugh, and Mass [20] and Cousins et. al [6] studied was 6/10" strand, and the results that they obtained for this strand are significantly different from those observed in the UTK tests. The 6/10" strands in UTK tests performed significantly better. In view of the limited amount of data on the 6/10" strand,

further experimental work with this strand would seem to be called for.

The transfer bond strengths for the UTK beams with milled surface 1/2" Regular, 1/2" Special, and 9/16" strands range from 1.48 to 2.26 times the strand diameter, with a mean of 1.84 times the strand diameter. The mean is within 1% of the H&K's transfer bond strength of 1.82 times the strand diameter. Therefore, the H&K's, the ACI, or the AASHTO equation for transfer length is supported by the UTK test results. The strands in H&K's research were gradually released of prestress, and confinement reinforcement was used in the UTK test beams; thus, the prediction equation based on the results of both the research might be less conservative for situations without any confinement reinforcement around multiple strands that are suddenly released of prestress. Because too little work has been done concerning the effects of confinement reinforcement, more research is recommended to attempt to substantiate the code provisions for transfer length.

The flexural bond strengths for all the strands in the UTK beam tests range from 0.306 to 0.517 times the strand diameter, with a mean of 0.428 times the strand diameter. Cousins's equation predicts the lowest flexural bond strength of about 0.262 times the strand diameter, while H&K's equation predicts the highest with a 0.608 times the strand diameter. The UTK mean flexural bond strength is close to the middle of the two extremes. This mean flexural bond strength is recommended until more data is available

to support the high bond strength that the ACI or the AASHTO equation permits. The recommended bond strength corresponds to an increase of $(0.608/0.428 = 1.42)$ about 42% for the flexural bond length portion of the code equation. The development length prediction equation therefore takes the following form:

$$L_d = (f_{se}/3)d_b + 1.42 (f_{ps} - f_{se})d_b \dots\dots\dots (7.1)$$

where:

L_d = development length (in),

f_{ps} = stress in prestressed reinforcement at nominal strength (ksi),

f_{se} = effective stress in prestressing steel after losses (ksi),

d_b = nominal diameter of prestressed reinforcement (in).

The development length according to equation (7.1) for all the strands in the UTK test program to develop the specified ultimate strength of 270 ksi from an effective of about 190 ksi is roughly 177 times the strand diameter, which is about a 23% increase in development length to the ACI prediction equation.

Chapter VIII

CONCLUSIONS

Considerable controversy has developed during the last few years as to the reliability of the code equations (ACI and AASHTO) for determining both transfer length and development length in prestressed concrete bridge beams. One purpose of this research was both to evaluate the research which has been done in the past and assess its relevance to what may actually be expected in a typical producer environment by testing full scale AASHTO Type I bridge beams. Also this research has attempted to identify and evaluate as many of the variables pertinent to transfer length and development length as possible.

Different definitions of transfer lengths have been used in the literature. As has been discussed in chapter IV, due to non-homogeneity in concrete, a limited precision of the instrumentation, and the smoothing effect from averaging strain measurement over a 10" gage length, there is a lack of a definite point in transfer strain-distribution diagram at which transfer length is determined. Much error in transfer length could be introduced from a misinterpretation for transfer length from the strain measurement. Noting the difficulty in practice of obtaining the actual transfer length, Stocker and Sozen [35] derived transfer length by multiplying by 1.12 the length from free end of a strand to the point of 90% of the concrete strain plateau. In their 1975 research, Hanson, Kaar, Corley, and

Hognestad [17] defined transfer length as the distance from the end of a strand to the point at which 95% of the concrete strain plateau was reached. Others have just presented their results for the theoretically defined transfer length. In this research, after considerable thought concerning this measurement, the *Slope-Intercept* method was developed to determine transfer lengths. The data presented in this report reflect this interpretation. Given this interpretation the transfer length results of this research compare favorably with the previous work of Hanson and Karr which is the basis of the current code equation (see Figure 7.1)

Based on the reviewed literature and the analysis of the UTK experimental results the following conclusions can be drawn:

- 1) The current ACI and AASHTO provisions for calculating transfer length are supported by the UTK test results, while the code flexural bond length requirement is significantly unconservative.

- 2) Transfer bond strength is directly related to strand diameter for strand ranging from 1/2" to 9/16" in diameter, but the larger 6/10" diameter strand did not follow this relationship; therefore more testing is required to determine the effects of larger strand on transfer length. Flexural bond strength is directly related to all strand diameters.

- 3) Strand surface condition is an important factor which ought to be considered in all future research. Weathered surface condition improves transfer and pullout bond strengths more significantly than flexural bond strength. The bond strength relationship found for

milled condition strand is a conservative lower bound solution, which is upper bound for the requirements of transfer and development lengths.

4) Concrete strength appears not to affect transfer and flexural bond strengths for 1/2" Regular, 1/2" Special, and 9/16" strands, but for the larger 6/10", the bond strength seems to be significantly affected by concrete strength. Also, there appears to be more scatter in the data for transfer bond strength at lower concrete strengths.

5) Transfer bond strength is virtually uninfluenced by the pretension levels from 175 ksi to 202.5 ksi. Therefore, the transfer length is directly proportional to steel stress in a strand.

6) For the limited data, transfer and flexural bond strengths appear not to be different for strands from different manufacturers. Reduction of strand spacing from 2" to 1.75" for the 1/2" strands did not appear to cause any adverse effects on the transfer and flexural bond strengths.

7) Average measured end slips of all strands in each beam at prestress transfer correlates well with the measured transfer lengths. Thus, measured slip would appear to be a very reliable quality control technique for precast producers. Care must be taken to "calibrate" slip measurements for a particular operation.

8) Flexural bond strength is correlated with transfer bond strength. The ratio of the transfer bond strength to the flexural bond strength is related to strand diameter.

9) Although different in magnitude, pullout bond strength is affected similarly to transfer bond strength by the variables of strand diameter, strand surface condition, and concrete strength. Transfer bond is most efficient, then pullout bond and then flexural bond.

10) Concrete prisms having a cross-sectional dimension of less than 3.5" may experience longitudinal cracking from sudden release of prestress force.

11) Development length of a beam that cracks in shear is more critical than an uncracked beam. The shear-cracked section may become a critical section for development length requirement evaluation.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Anderson, A.R., and Anderson, R. G., (1976). "An Assurance a Criterion for Flexural Bond in Pretensioned Hollow Core Units," *Journal of American Concrete Institute*, 73(8),457-464.
2. Brearley, L.M., and Johnston, D.W., "Pull-Out Bond Tests of Epoxy-Coated Prestressing Strand," *Journal of Structural Engineering*, August 1990.
3. Brooks, M.D., Gerstle, K.H., and Logan, D.R., (1988). "Effect of Initial Strand Slip on the Strength of Hollow Core Slabs," *Journal of Prestressed Concrete Institute*, 33(1), 90-111.
4. "Building Codes Requirements for Reinforced Concrete," (1983). ACI 318-83, *American Concrete Institute*, Detroit, Michigan.
5. Burnett, Eric F.P., and Ahmed H. Anis, "Bond Characteristics of Initially Untensioned Strand," *Journal of the Structural Division, ASCE*, Vol. 107, May 1981.
6. Cousins,T., Johnston, D., and Zia, P. (1986). "Bond of Epoxy Coated Prestressing Strand," Department of Civil Engineering, North Carolina State university at Raleigh.
7. Cousins, T.E., Johnston, D.W., and Zia, P., "Transfer Length of Epoxy-Coated Prestressing Strand," *ACI Materials Journal*, May-June 1990.
8. Cousins, T.E., Johnston, D.W., and Zia, P., "Transfer Length of Epoxy-Coated Prestressing Strand," *ACI Materials Journal*, July-August 1990.
9. Deatherage, J.H. (1988). "Investigation of Physical Properties of Concrete At Early Ages," Department of Civil Engineering, University of Tennessee, Knoxville.

10. Deatherage, J.H., and Burdette, E.G. (1990), "Development Length and Lateral Spacing Requirements of Prestressing Strand for Prestressed Concrete Bridge Products," Draft Final Report, The University of Tennessee, Knoxville.
11. Dorsten, V., Hunt, F.F., and Preston, H.K. (1984). "Epoxy Coated Seven-Wire Strand for Prestressed Concrete," *Journal of Prestressed Concrete Institute*, 29(4), 121-128.
12. Edwards, A.D., and Picard A., "Bonding Properties of 1/2 in. Diameter Strand," *Journal of American Concrete Institute*, Nov. 1972, p.684-689.
13. Ghosh, S.K., and Fintel, M. (1986). "Development Length of Prestressing Strand, Including Debonded Strands, and Allowable Concrete Stresses in Pretensioned Members," *Journal of Prestressed Concrete Institute*, 31(5),38-57.
14. Griffin, Ken L., "An Investigation of the Variables Affecting Pullout of Seven-wire Prestressing Strand," M.S. Thesis, The University of Tennessee at Knoxville, May 1989.
15. Hanson, N.W., and Kaar, P.H. (1959). "Flexural Bond Tests of Pretensioned Prestressed Beams," *Journal of American Concrete Institute*, 55(7), 783-803.
16. Hanson, N.W., "Influence of Surface Roughness of Prestressing Strand on Bond Performance," *Journal of Prestressed Concrete Institute*, Feb. 1969.
17. Hanson, N.W., Kaar, P.H., Corley, W., and Hognestad, E. (1975). "Bond Fatigue Test of Pretensioned Concrete Cross-Ties," *Journal of Prestressed Concrete Institute*, 20(5), 56-72.
18. Janney, J.R. (1954). "Nature of Bond in Pre-Tensioned, Prestressed Concrete," *Journal of American Concrete Institute*, 50, 717-736.

19. Janney, J.R. (1963). "Report of Stress Transfer Length Studies on 270K Prestressing Strand," *Journal of Prestressed Concrete Institute*, 8(1), 41-43.
20. Kaar, P.H., LaFraugh, R.W., and Mass, M.A. (1963). "Influence of Concrete Strength on Strand Transfer Length," *Journal of Prestressed Concrete Institute*, 8(5), 47-67.
21. Kaar, P.H., and Magura, D.D. (1965). "Effects of Strand Blanketing on Performance of Pretensioned Girders," *Journal of Prestressed Concrete Institute*, 10(6), 20-34.
22. Keuning, R.W., "Progress Report on the Experimental Study of Bond Characteristics of Strand Reinforcement," (Tenth Progress Report, Investigation of Prestressed Reinforced Concrete for Highway Bridges, Part 4), University of Illinois, Oct. 1961.
23. Laldji, S. and Young A.G., "Bond between Steel Strand and Cement Grout in Ground Anchorages," *Magazine of Concrete Research*, Vol. 40, No.143: June 1988.
24. Lin, T.Y., and Burns, N.H. (1981). *Design of Prestressed Concrete Structures*, John Wiley and Sons.
25. Martin, L.D., and Scott, N.L. (1976). "Development of Prestressing Strand in Pretensioned Members," *Journal of American Concrete Institute*, 73(8), 453-456.
26. Maruyama, Kyuichi, and Rizkalla, Sami H., "Shear Design Consideration for Pretensioned Prestressed Beams," *ACI Structural Journal*, Sept-Oct. 1988, p.492-498.
27. Over, R.S., Au, T. (1966). "Prestress Transfer Bond of Pretensioned Strands in Concrete," *Journal of American Concrete Institute*, 62, 1451 -1460.
28. "PCI Design Handbook," (1985). *Prestressed Concrete Institute*, Chicago, Illinois.

29. Presswalla, H., and Preston, H.K., "Torque in Tensioned Strand," *Journal of Prestressed Concrete Institute*, March-April 1990.
30. Preston, H.K., (1988). "Bond of Seven-Wire Strand," Technical Bulletin, Wiss, Janney, Elstner, Associates, Inc., Princeton Junction, NJ.
31. Salmon, John R., and McCrate, Timothy E., "Bond Characteristics of Untensioned Prestressing Strand," *Journal of Prestressed Concrete Institute*, Jan.-Feb. 1977.
32. Scribner, C.F., and Kobayashi, K. (1984). "Prestressing Strand Bond Characteristics Under Reverse Cyclic Loading," *Journal of Prestressed Concrete Institute*, 29(5), 118-137.
33. Smith, B.R., "An Investigation of the Variables Affecting the Transfer Length of Prestressed Concrete Members," M.S. Thesis, The University of Tennessee, Knoxville, Dec. 1989.
34. "Standard Specifications for Highway Bridges" (1983a). American Association of State Highway Transportation Officials, Washington, DC.
35. Stocker, M.F., and Sozen, M.A., "Bond Characteristics of Prestressing Strand," (Part V of Investigation of Prestressed Reinforced Concrete for Highway Bridges), Engineering Experiment Station, Bulletin 503, University of Illinois, Urbana, 1970.
36. Tulin, Leonard G., and Al-Chalabi, Maluf M., "Bond Strength as a Function of Strand Tension and Cement Paste Content for Lightweight Aggregate Concrete," *Journal of American Concrete Institute*, Oct. 1969, p.840-846.
37. Warwaruk, J., Sozen, M.A., and Siess, C.P., "Strength and Behavior in Flexure of Prestressed Concrete Beams," Engineering Experiment Station Bulletin No. 464, University of Illinois, 1962.

38. Wang, Chu-Kia, and Salmon, C.G. (1985). *Reinforced Concrete Design*, Harper and Row.
39. Zia, P., and Mostafa, T. (1977). "Development Length of Prestressing Strands," *Journal of American Concrete Institute*, 22(5), 54-65.

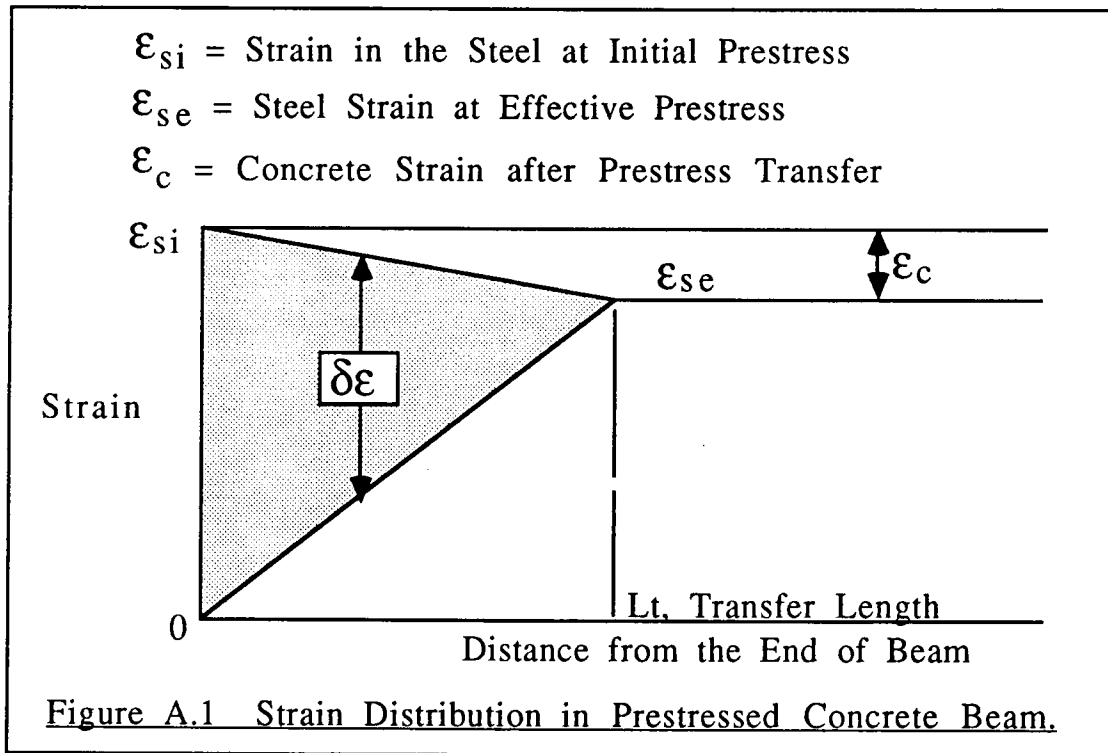
APPENDICES

APPENDIX A

THEORETICAL CALCULATIONS

A.1 Theoretical End Slip at Prestress Transfer

Based on a constant transfer bond distribution within the transfer length, the cumulative slip at the end of a beam is the difference in the shortening of the strand and the concrete over the entire transfer length. This difference in axial deformation at any point along the beam is graphically depicted in Figure A.1. The shaded area is the cumulative end slip.



Cumulative End Slip, Δ = Shaded Area

$$= \frac{\epsilon_{si} * Lt}{2} = \frac{f_{si}}{2 * E_s} * Lt$$

Or,

$$Lt = \frac{2 * E_s * \Delta}{f_{si}} = \frac{2 * 27800 \text{ ksi} * \Delta}{202.5 \text{ ksi}}$$

$$Lt = 275 * \Delta$$

A2 Relationship between the Area and the Diameter of Strand

The area of seven-wire prestressing strand is proportional to the square of the strand diameter as shown in Figure A2. The coefficient 0.608 can also be obtained theoretically. The actual area of the strand is reduced to 7/9 of a circular area with the strand diameter.

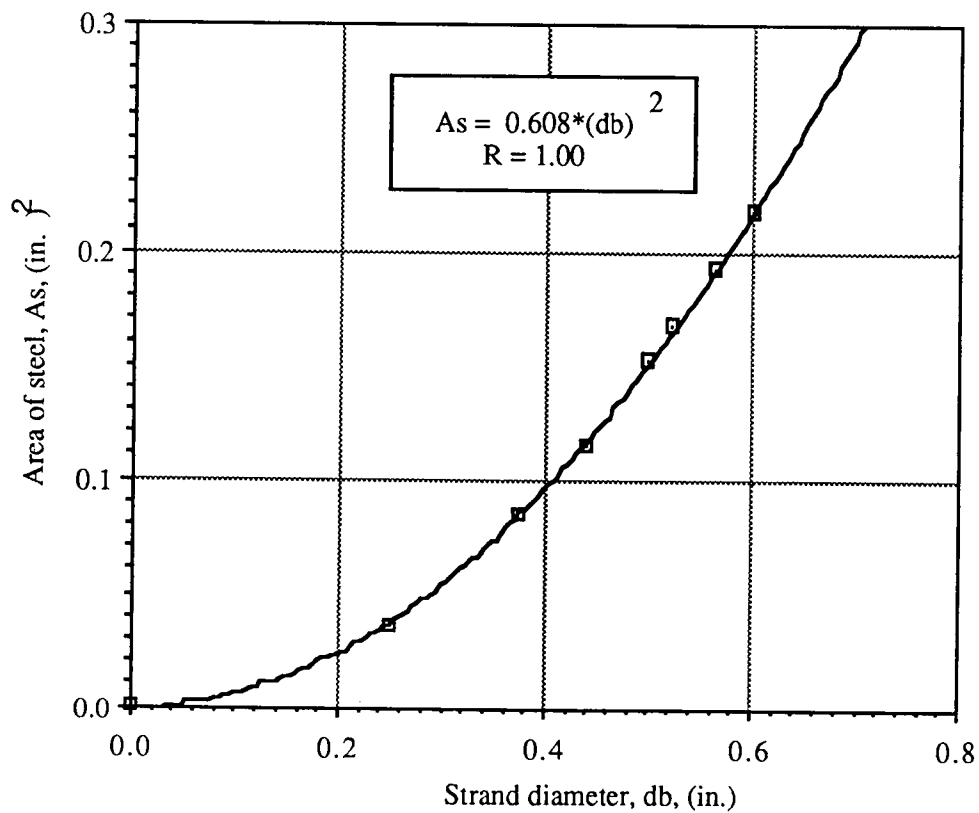


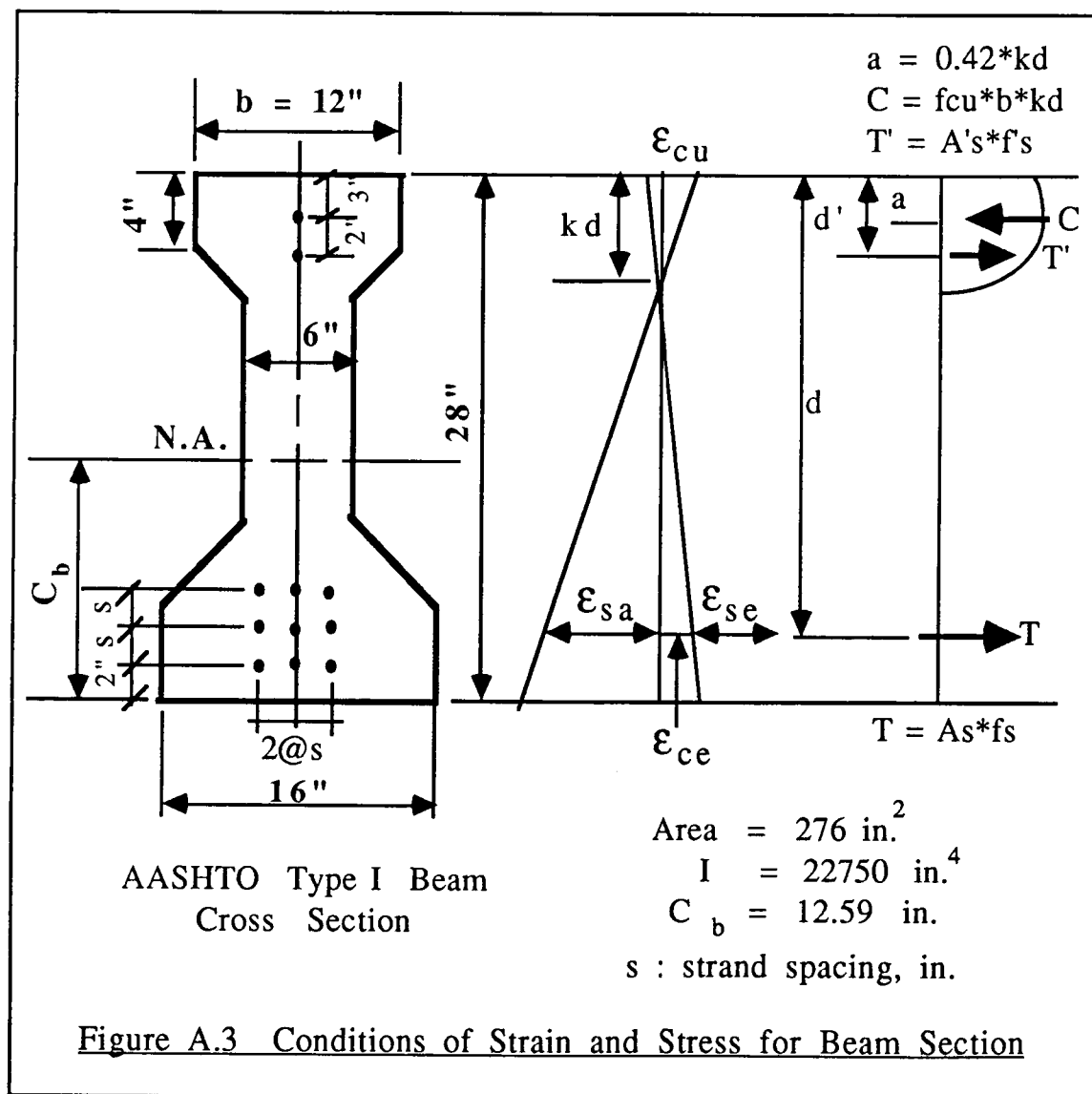
Figure A.2 Relationship between the Area and the Diameter of Strand

A.3 Theoretical Steel Stress at Ultimate Flexural Compression Failure

The ultimate moment strength of each beam was measured during the development length tests. Due to confinement vertically and laterally by the steel bearing plate at the load point of the top fibers of the concrete in compression and the sharp stress gradient beneath the single load point, a considerably higher concrete strength was apparently attainable at the critical load-point section. The moment strength calculated according to the ACI allowable rectangular stress block and strain-compatibility method is therefore significantly lower than the actual moment strength for these beams. The steel stress will be overestimated greatly if it is to be calculated from the measured moment strength using the ACI method. This overestimation will in turn lead to an error in the steel stress assumed to have been developed which is then used in the derivation of the bond strength for the development length. While the ACI method for the calculation of flexural strength is adequate for general design purposes, particularly for a flexural member without any lateral confinement for the concrete in compression, it is not sufficiently accurate for the needs of the present research.

A more fundamental approach somewhat similar to the analysis method found in the publication by Warwaruk, et. al. [37] is devised here to solve this problem. In their research, the compressive concrete in the beams loaded at mid-span appeared to have double the effective strength of the concrete in beams with

two-point loadings. Typically, a condition of strain and stress at failure is assumed for a beam as in Figure A.3. The steel strain at ultimate is calculated by adding together the steel strain at effective prestress, ϵ_{se} , the elastic shortening, ϵ_{ce} , and the additional steel strain beyond flexural cracking of the section, ϵ_{sa} . Assumptions for the present analysis include:



- 1) plane section remains plane under flexure,
- 2) strain compatibility factor is known and is a constant 1.0,
- 3) failure of concrete occurs when concrete strain in the extreme fibers reaches a useful limit of 0.005, i.e. $\epsilon_{cu} = 0.005$.
- 4) the location of the resultant compressive force of the concrete is at a distance from the extreme compressive fibers equal to 0.42 of the depth of the neutral axis, i.e., $a = 0.42 \cdot kd$.
- 5) concrete does not carry tension.
- 6) the stress-strain relationship for the reinforcement is known.
- 7) a horizontal inward reaction, H, equal to a friction coefficient of 0.4 times the smaller of the two support reaction forces.

It is realized that the assumption (4) above is likely to vary very slightly with concrete strength or when kd is greater than the flange thickness of 4". However, the calculated flexural strength is quite insensitive to such small variation. For example, for an actual value of $a = 0.35 \cdot kd$ in comparison to the assumed $a = 0.42 \cdot kd$, only an error of 0.35" in moment arm results for a typical kd value of 5". This translates into an error in the flexural strength of less than 1.5%.

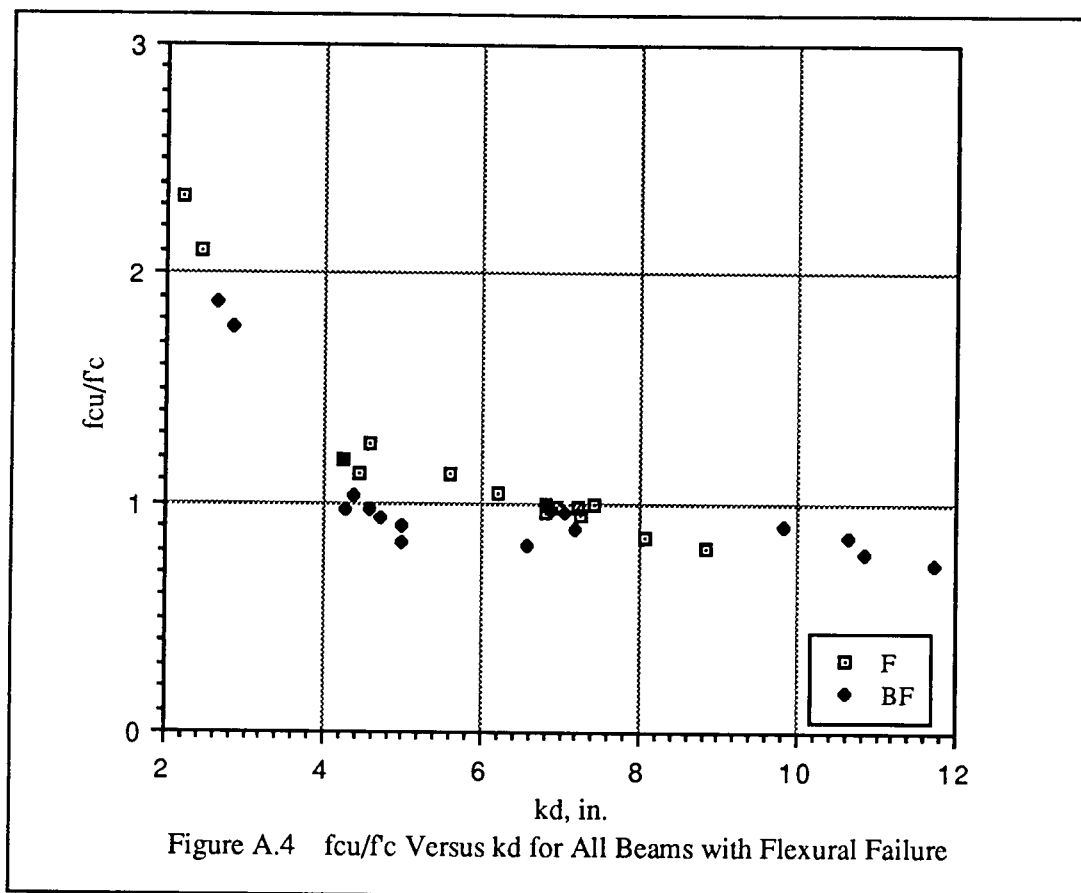
It is necessary to determine the effective concrete strength, f_{cu} , of the concrete subjected to the effect of confinement for the beams in this test program. A computer program was written which iteratively calculates the f_{cu} for each of the beams from the measured ultimate moment in accordance with the strain compatibility condition for the steel strand and the assumptions stated previously. The calculation is done as follows:

- 1) set $\epsilon_{cu} = 0.005$,
- 2) assume a kd value,
- 3) set $a = 0.42 \cdot kd$,
- 4) calculate the steel strain, and the corresponding steel stress,
- 5) calculate the moment strength, M_n ,
- 6) when M_n is within 0.1% of M_u , go to step (9),
- 7) if M_n is less than the measured moment strength, M_u , then
decrease kd , and go to step (3),
- 8) if M_n is greater than M_u then increase kd , and go to step (3),
- 9) calculate effective concrete strength, f_{cu} , and stop iteration.

The ratio of the effective concrete strength to the 6" x 12" concrete cylinder strength, f_{cu}/f'_c , is calculated. The results of the above calculation are summarized in Table A.1 under the heading of "With Strain Compatibility." The f_{cu}/f'_c is plotted against the depth to the neutral axis, kd , in Figure A.4. From this figure, the ratio of f_{cu}/f'_c is much larger at smaller values of kd and rapidly reduces to a more stable lower value of approximately 0.9 as kd value increases. The data points with the smaller values of kd correspond to the beams with the smaller reinforcement ratios. The larger f_{cu}/f'_c with smaller kd values is consistent with the knowledge that the effect of confinement is more significant right beneath the bearing plate and within a shallower kd .

TABLE A.1 Summary of f_{cu}/f'_c and Steel Stress Calculations

Specimen Designation	Concrete Strength f'_c (ksi)	Horizontal Reaction H, Kips	Measured Moment Mn, (k-ft)	Mode of Failure	With Strain Compatibility			Force Equilibrium		
					Steel Stress f_s (ksi)	Depth to N.A. kd (in.)	Calculated f_{cu}/f'_c	Adjusted f_{cu}/f'_c	Calculated kd (in.)	fsc (ksi)
5-1-EXT	5.48	12.7	645	B-F	277	6.20	1.05	1.20	5.13	270
5-1-INT	5.48	10.6	564	B-S	273	11.18	0.69	1.20	4.49	232
5-2-EXT	6.75	11.6	612	B-S	276	8.28	0.70	1.20	3.90	248
5-2-INT	6.75	12.3	639	B-F	277	6.57	0.81	1.20	4.04	260
5-3-EXT	6.86	12.9	675	F	278	4.44	1.13	1.20	4.17	277
5-3-INT	6.86	12.7	679	B-F	279	4.23	1.19	1.20	4.19	279
5-4-EXT	7.60	12.8	676	F	278	4.37	1.03	1.20	3.77	274
5-4-INT	7.60	12.5	665	B-F	278	4.99	0.91	1.20	3.72	269
5-SWAI-EAST	5.55	12.2	641	F	277	6.81	0.96	1.20	4.96	265
5-SWAI-WEST	5.55	12.2	660	F-B	277	5.58	1.12	1.20	5.12	275
5-UWR-EAST	5.99	12.7	679	B-F	278	4.57	1.26	1.20	4.81	280
5-UWR-WEST	5.99	20.7	707	B-F	279	4.01	1.47	1.20	5.00	286
5-FWC-EAST	5.34	11.9	641	B-F	277	6.81	1.00	1.20	5.19	267
5-FWC-WEST	5.34	12.4	660	F	277	5.65	1.15	1.20	5.37	276
5-ASW-EAST	5.40	11.9	633	F	277	7.27	0.95	1.20	5.06	263
5-ASW-WEST	5.40	11.8	638	F	277	6.93	0.98	1.20	5.11	265
5S-1-EXT	6.62	10.7	569	B-S	256	15.83	0.51	0.90	5.19	196
5S-1-INT	6.62	13.6	724	F-B	268	8.08	0.85	0.90	7.26	263
5S-2-EXT	-	-	-	-	-	-	-	-	-	-
5S-2-INT	-	-	-	F-B	-	-	-	0.90	5.95	223
5S-3-EXT	5.97	12.2	640	F	268	8.84	0.80	0.90	7.01	257
5S-3-INT	5.97	11.1	587	B-S	263	11.66	0.65	0.90	6.14	231
5S-4-EXT	6.18	10.7	574	B-S	262	12.32	0.60	0.90	5.67	223
5S-4-INT	6.18	12.3	666	B-F	269	7.19	0.89	0.90	7.03	268
916-1-EXT	5.53	14.0	688	B-F	261	10.63	0.85	0.90	9.56	254
916-1-INT	5.53	13.5	703	B-F	262	9.85	0.90	0.90	9.95	263
916-2-EXT	5.92	13.2	666	B-F	260	11.71	0.74	0.90	7.85	237
916-2-INT	5.92	13.1	682	B-F	261	10.86	0.78	0.90	8.19	245
916-3-EXT	6.12	13.1	663	B-F	274	2.85	1.76	2.20	2.31	271
916-3-INT	6.12	13.7	675	F	277	2.45	2.10	2.20	2.34	276
916-4-EXT	6.24	13.6	669	B-F	275	2.66	1.88	2.20	2.28	272
916-4-INT	6.24	14.0	684	F	278	2.20	2.33	2.20	2.34	281
6-1-EXT	5.13	13.9	658	F	264	7.22	0.98	1.00	6.91	262
6-1-INT	5.13	27.4	682	F	263	7.44	1.00	1.00	7.37	263
6-2-EXT	5.29	12.6	660	B-F	264	6.89	0.98	1.00	6.57	262
6-2-INT	5.29	12.3	657	B-F	264	7.06	0.96	1.00	6.51	261
6-3-EXT	7.46	13.3	701	B-F	268	4.72	0.94	1.00	4.42	266
6-3-INT	7.46	13.6	704	F	268	4.59	0.97	1.00	4.44	268
6-4-EXT	7.98	13.3	695	F	267	5.00	0.83	1.00	4.08	262
6-4-INT	7.98	13.6	711	F	269	4.29	0.97	1.00	4.16	269
5E-1-EXT	5.78	12.2	684	F	279	3.88	1.55	1.20	5.11	288
5E-1-INT	5.78	12.3	665	F	278	4.93	1.21	1.20	4.96	278
5E-2-EXT	6.57	10.9	620	B-S-F	277	7.65	0.75	1.20	4.04	253
5E-2-INT	6.57	11.5	652	F	277	5.63	0.94	1.20	4.22	268



The figure also shows that for the beams with bond failure, the data points for which are plotted as BF, the f_{cu}/f_c ratio is generally smaller than for the beams without bond failure (F). This is, however, an inherent result of the assumption for the analysis that strain compatibility holds and of the fact that the beams with bond failure generally have slightly lower moment strengths. For the beams with bond failures, deviation from the strain compatibility condition leads to a kd value greater than it would have been with no bond failure. This greater kd value consequently enters into the calculation of the resulting lower f_{cu} value. Thus, only the beams which develop the full flexural strengths without any end slippages

are used in establishing the f_{cu} needed for the calculation of the steel stresses developed in the beams with bond failure.

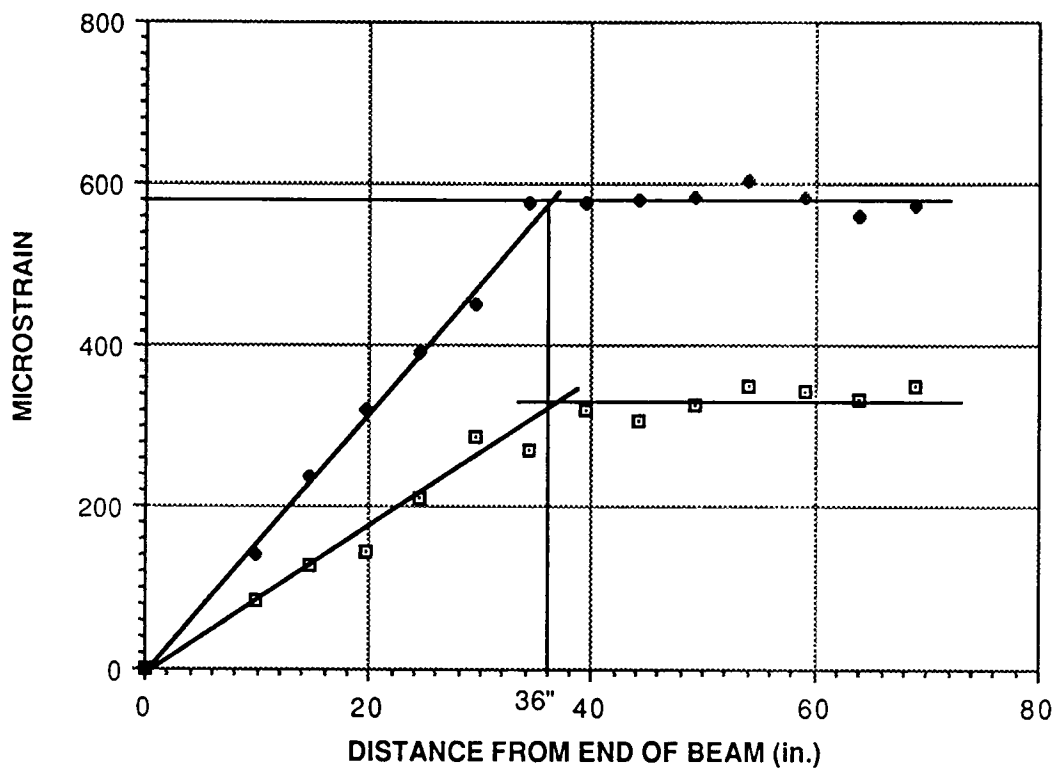
For the beams with bond failure, the steel stresses can be calculated from the measured moment strengths with a chosen approximate f_{cu} value for each beam group in reference to the f_{cu}/f'_c calculated earlier with strain compatibility for beams without bond failure. Because the bottom strands in the beams with bond failure slip, the assumption of strain compatibility condition is not valid. Except the strain compatibility condition for the bottom steel, the assumptions for the flexural strength analysis stated previously are used. The top steel does not slip; its stress can therefore be calculated from strain compatibility with the previously set $\epsilon_{cu} = 0.005$ and a k_d value. The concrete force, C , and position, $a = 0.42 \cdot k_d$, from which the moment, M_n , can be calculated, are only dependent on the k_d value. There is a unique k_d value which gives the M_n that is equal to the measured moment. A computer program was written to iteratively seek for this k_d value. The steel stress can then be calculated for this k_d from the equilibrium of horizontal forces. The calculated steel stresses and the corresponding adjusted f_{cu} values are presented in Table A.1 under the heading of "With Force Equilibrium."

The steel stress of the beams which failed by shear is not predictable with the flexural analysis. These beams are shaded in Table A.1.

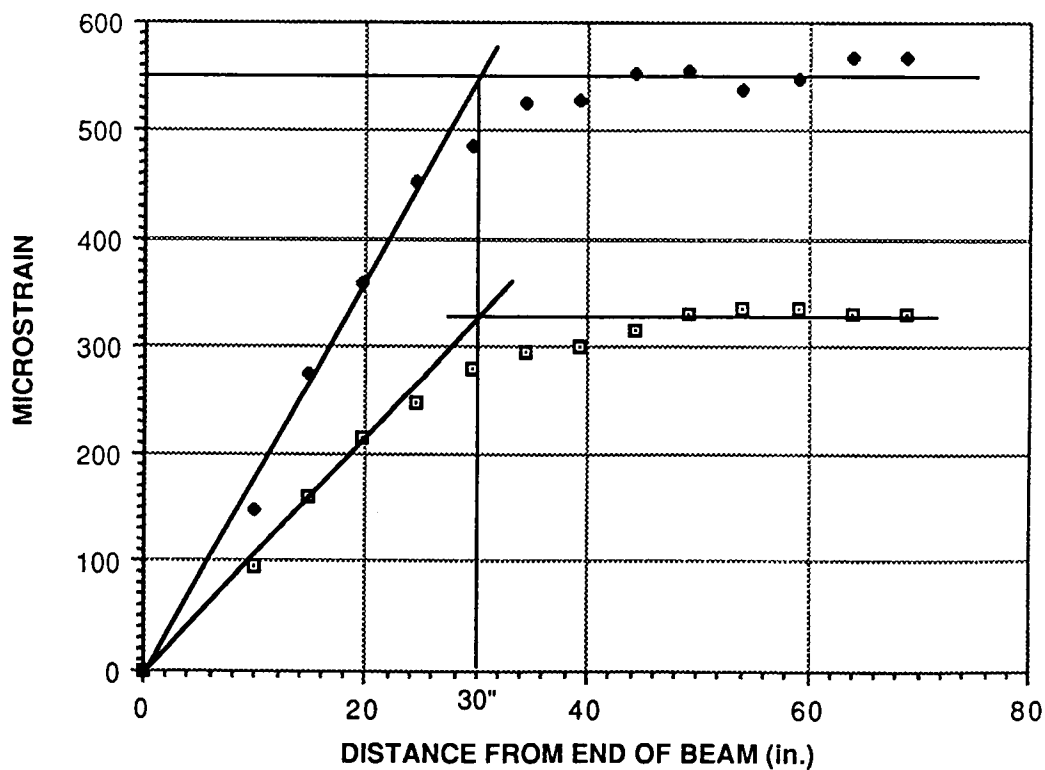
APPENDIX B

**MEASURED TRANSFER
STRAIN-DISTRIBUTION
DIAGRAMS**

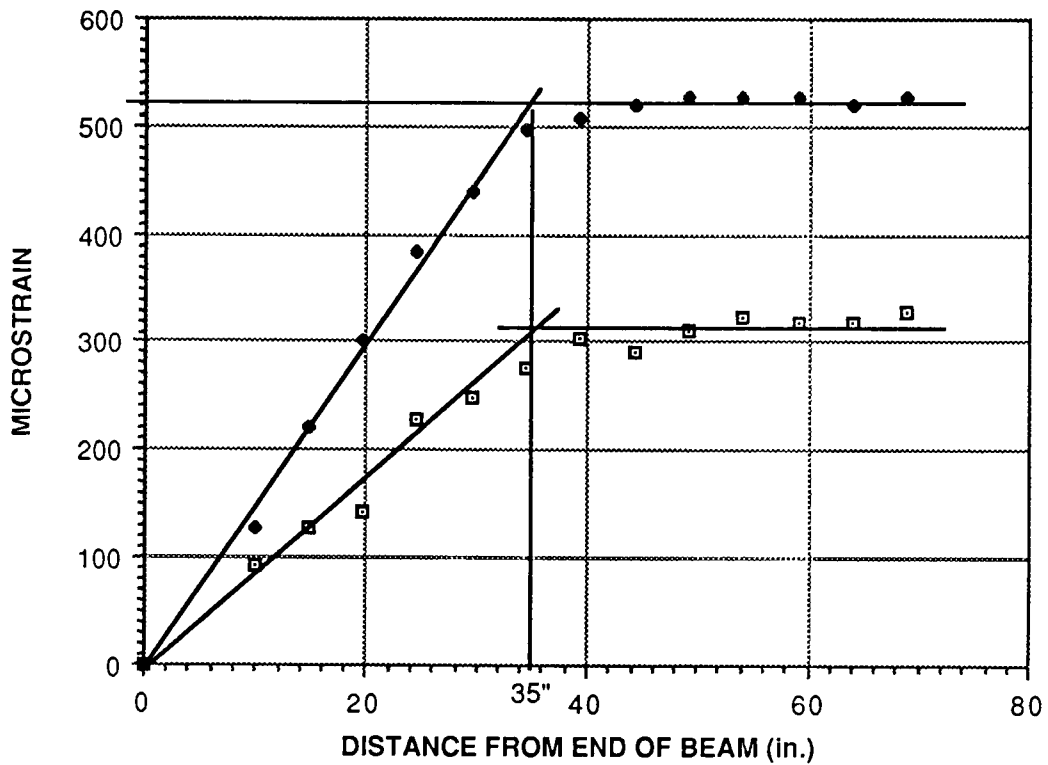
TRANSFER BEAM 5-1-EXT



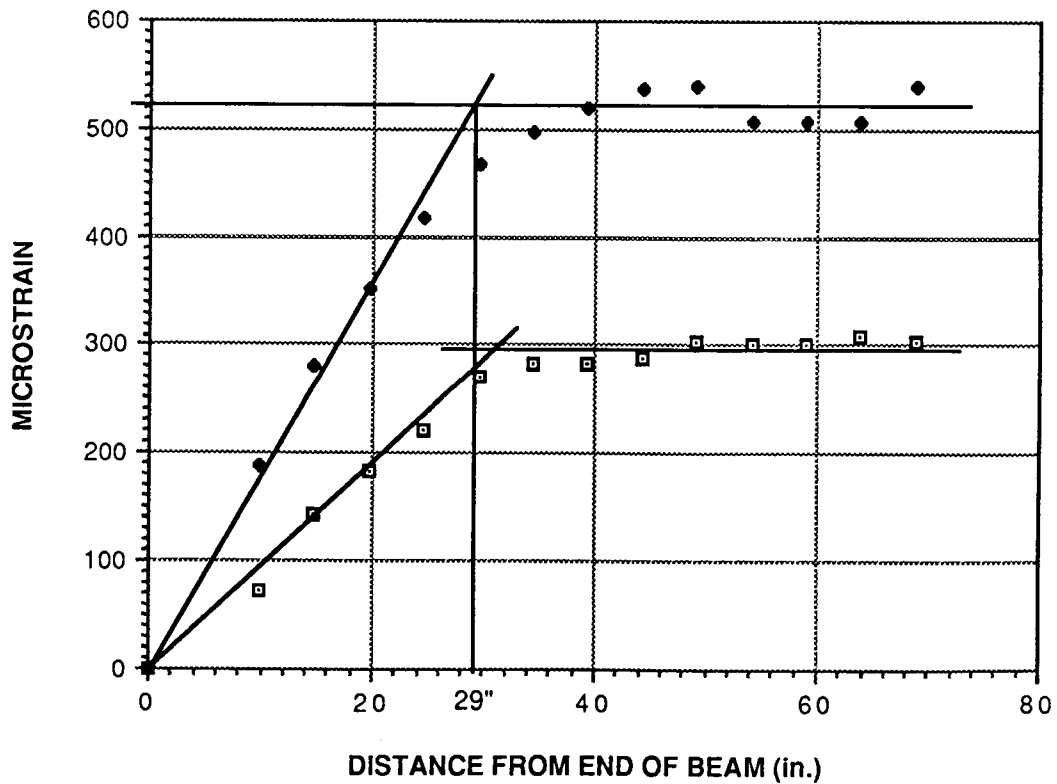
TRANSFER BEAM 5-1-INT

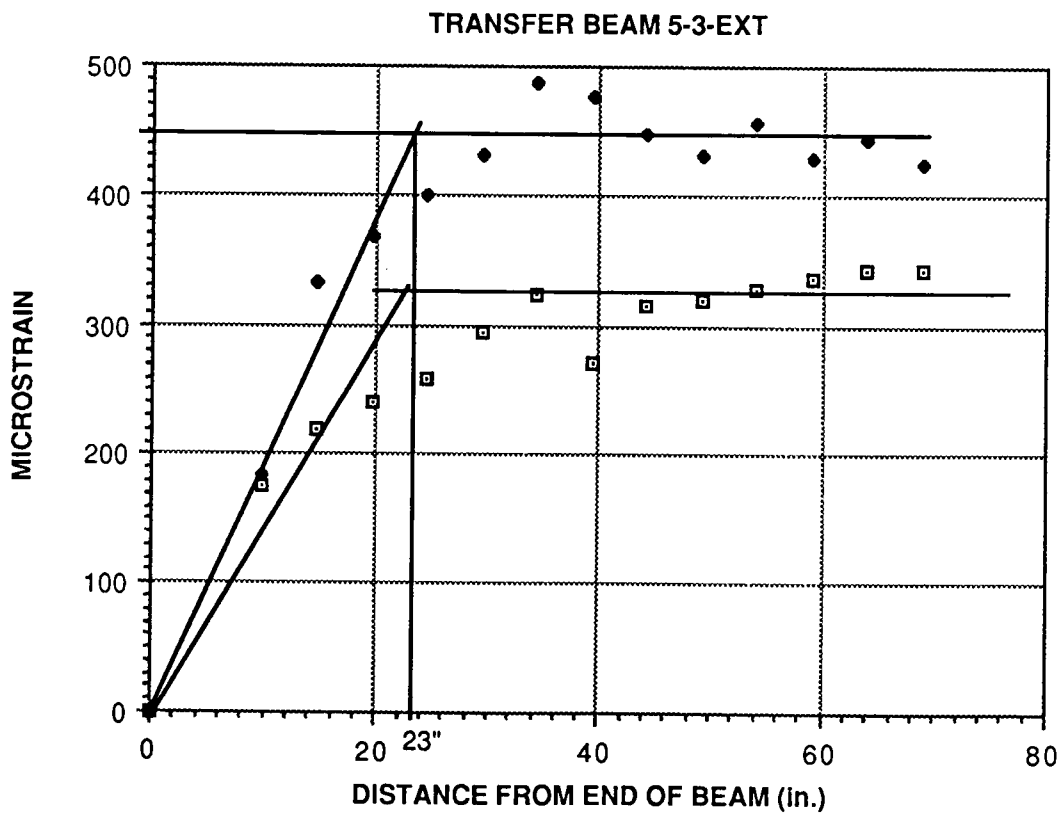
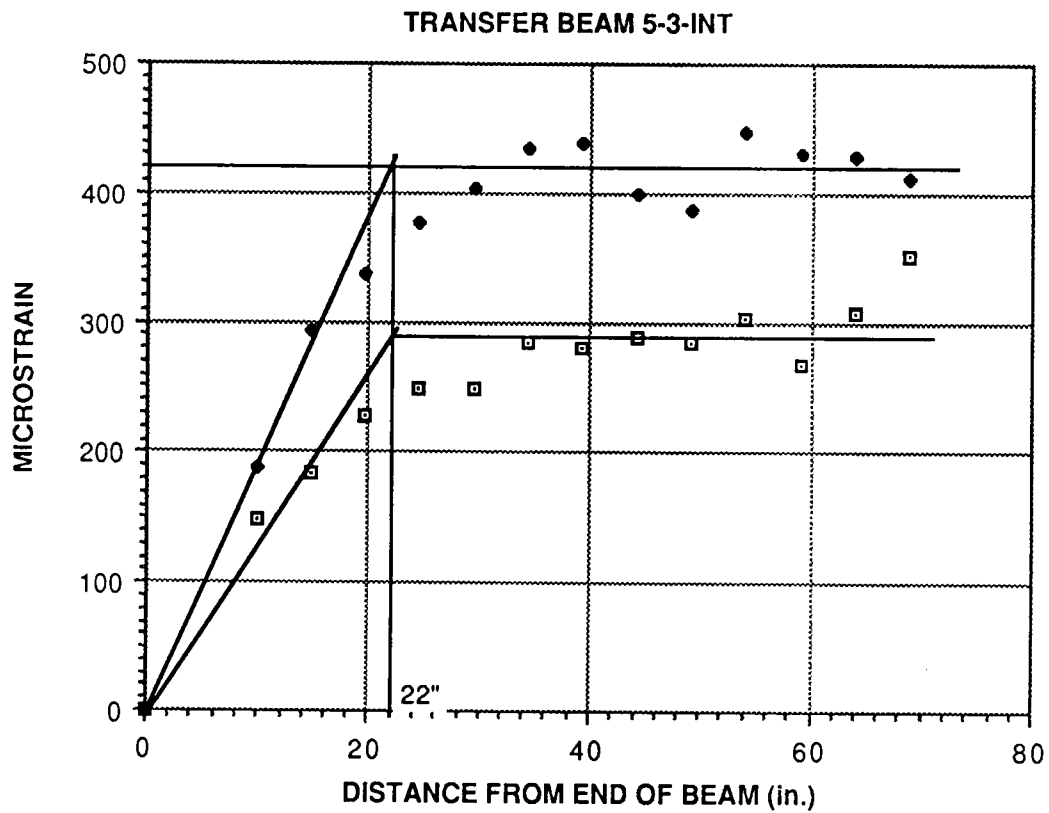


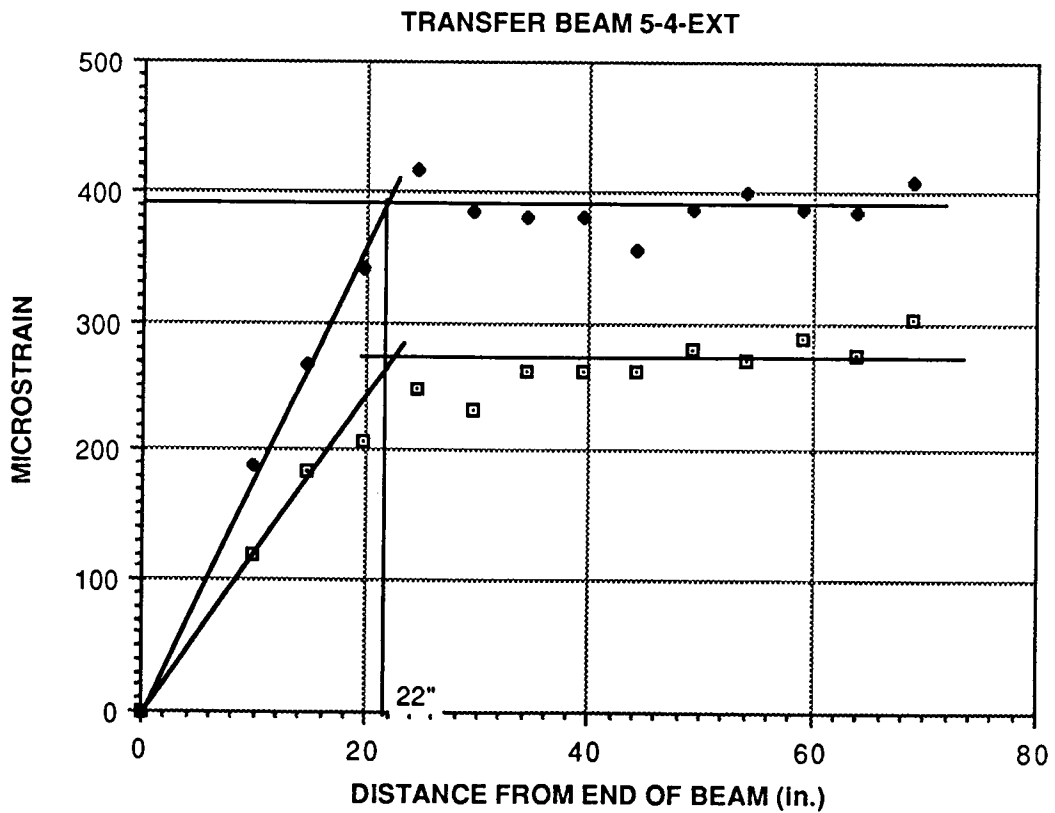
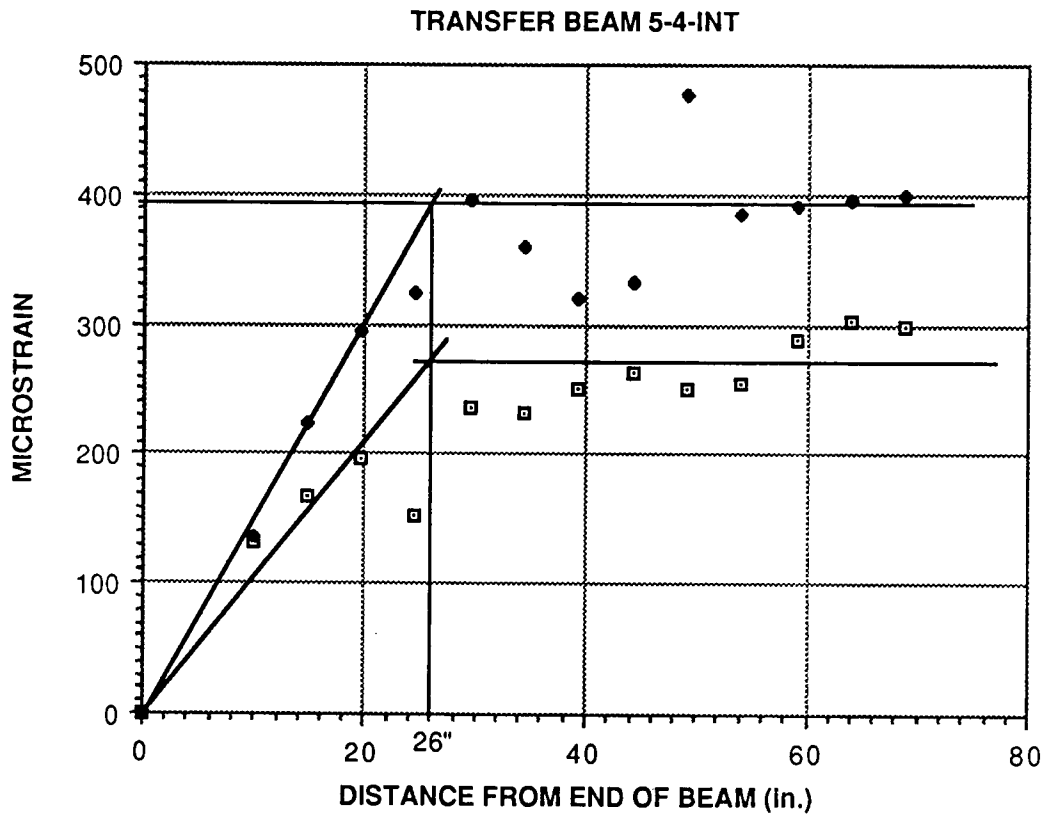
TRANSFER BEAM 5-2-EXT



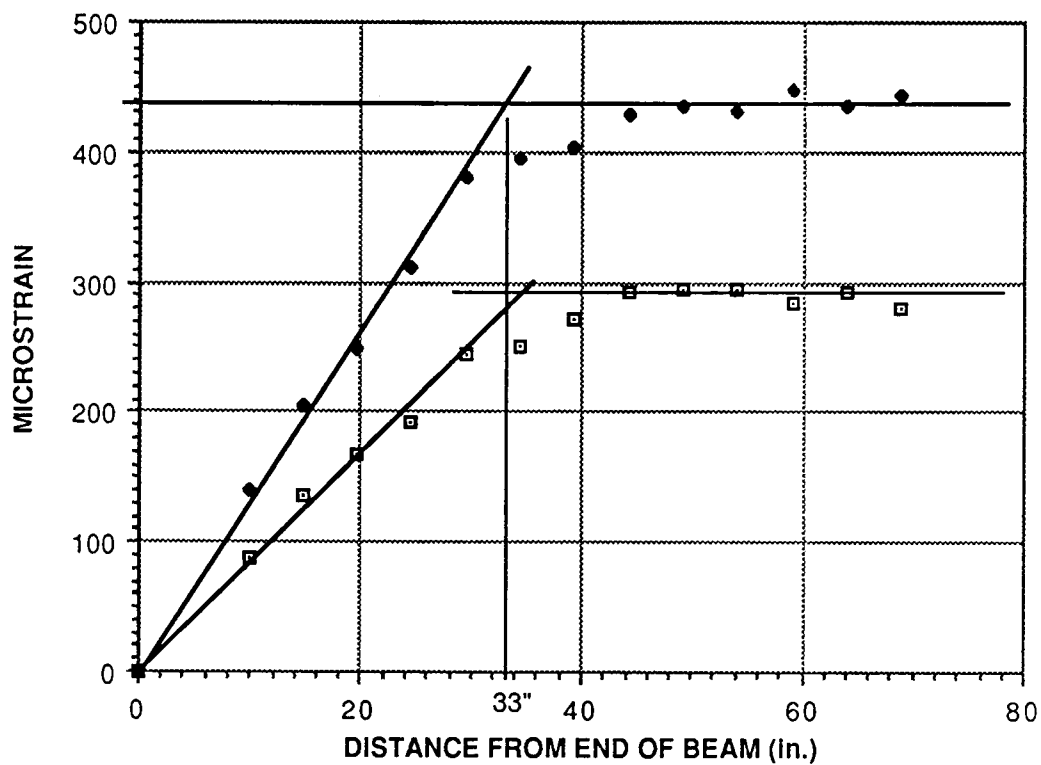
TRANSFER BEAM 5-2-INT



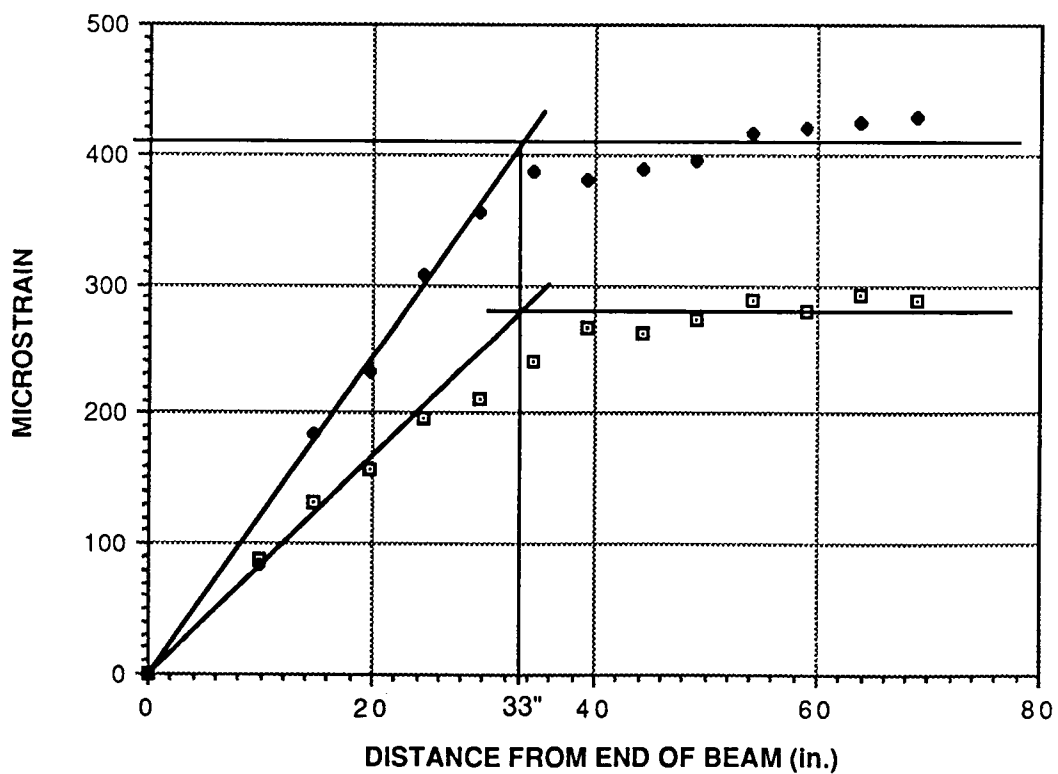


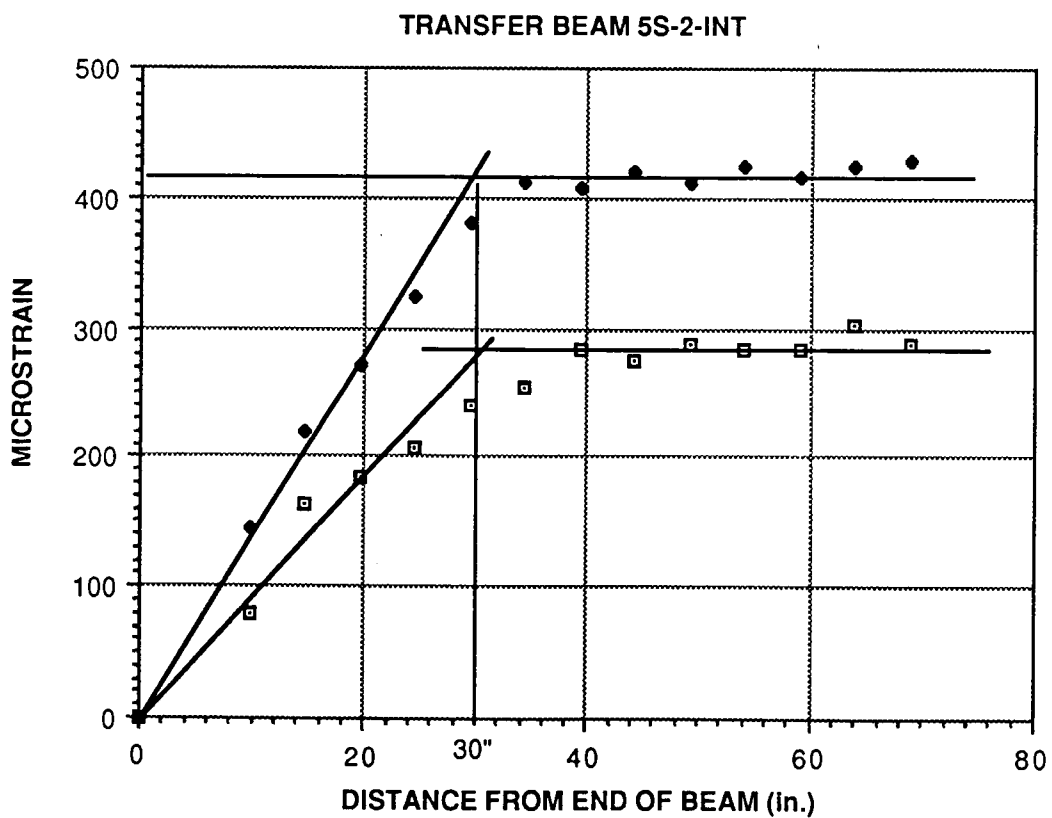
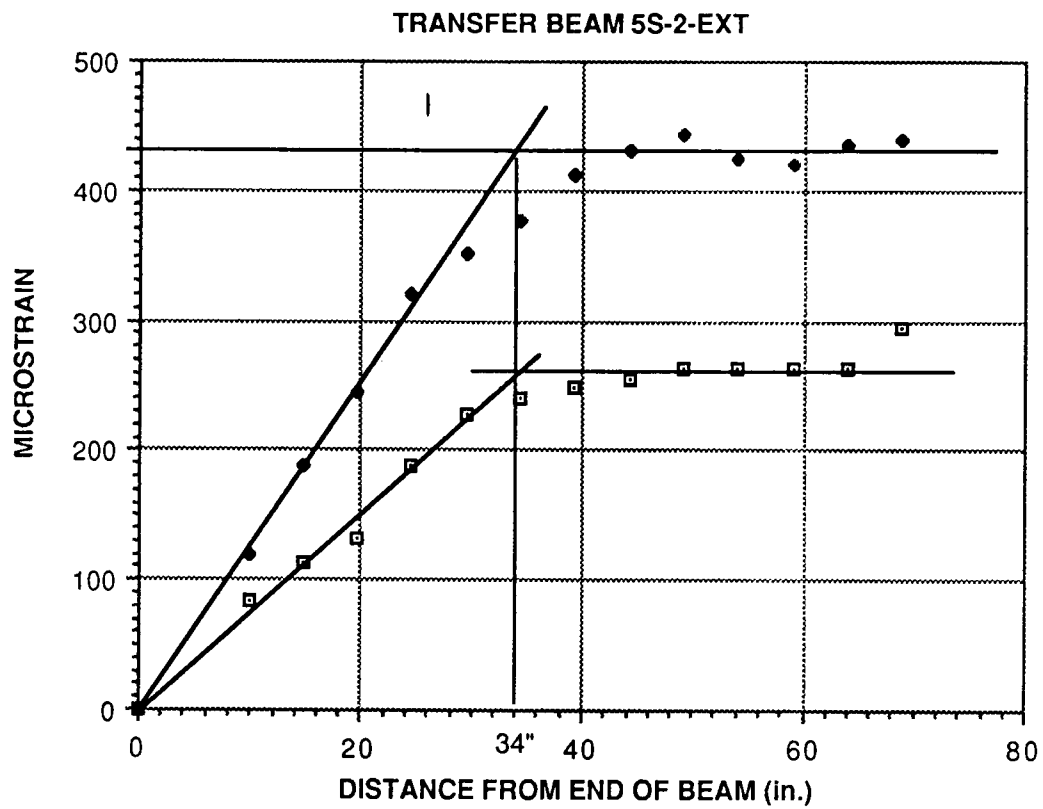


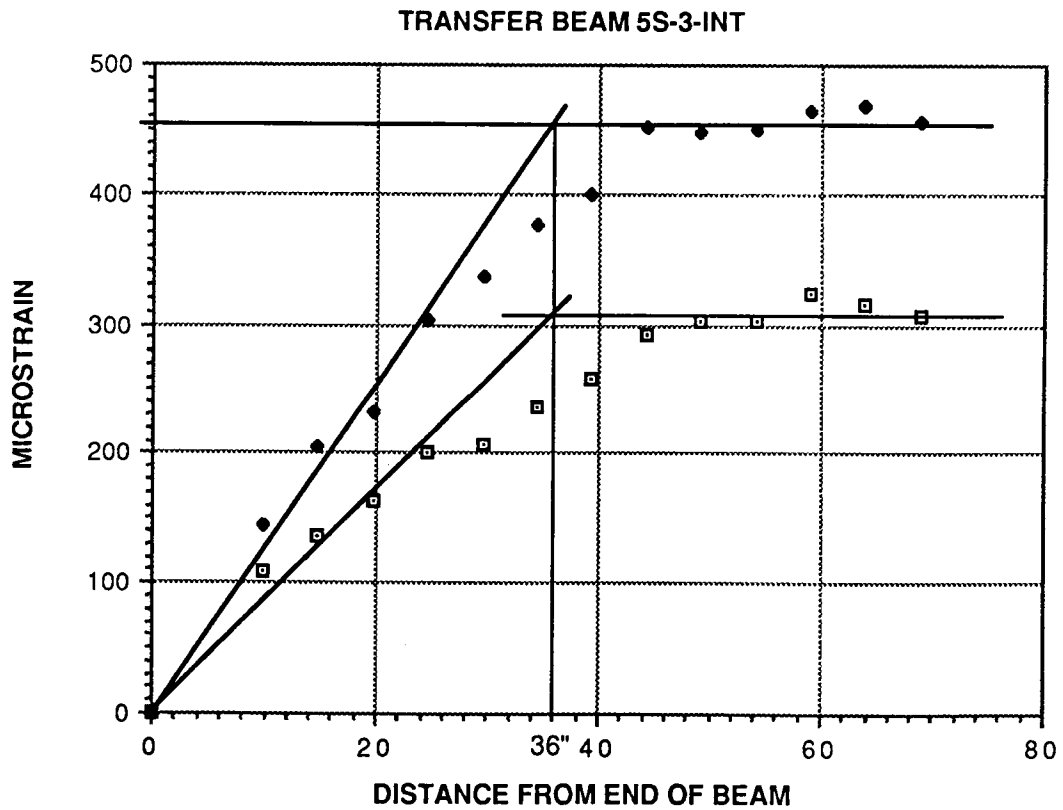
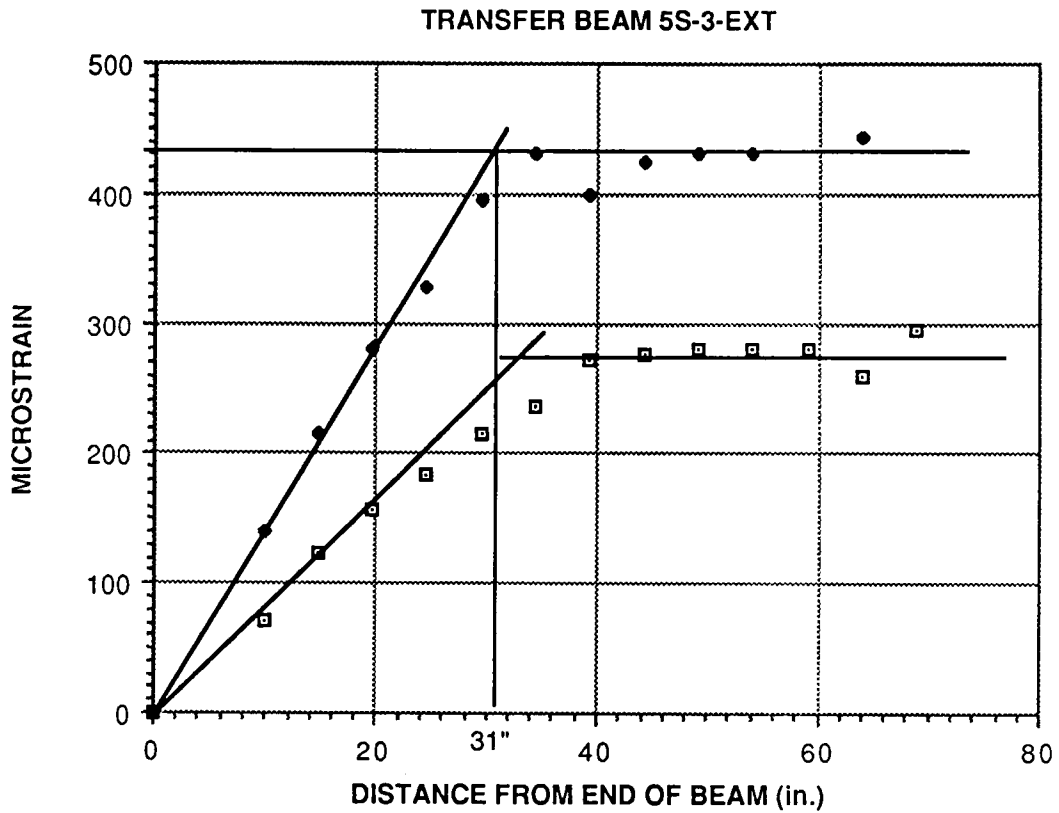
TRANSFER BEAM 5S-1-EXT

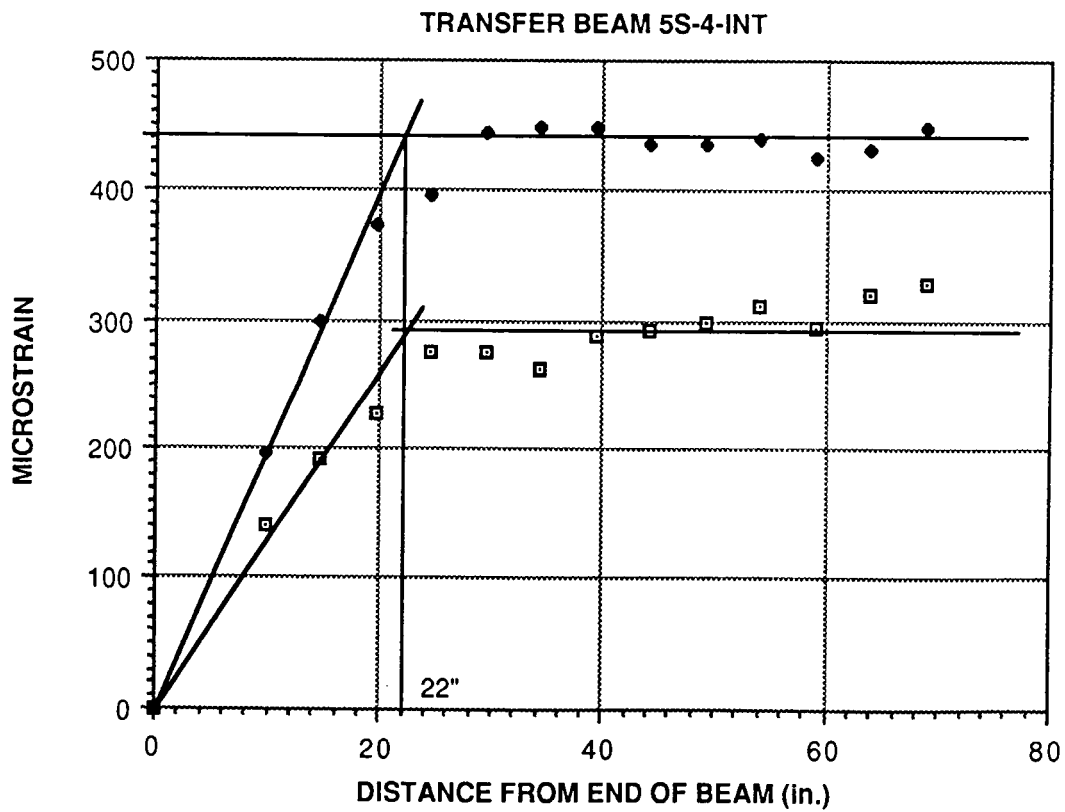
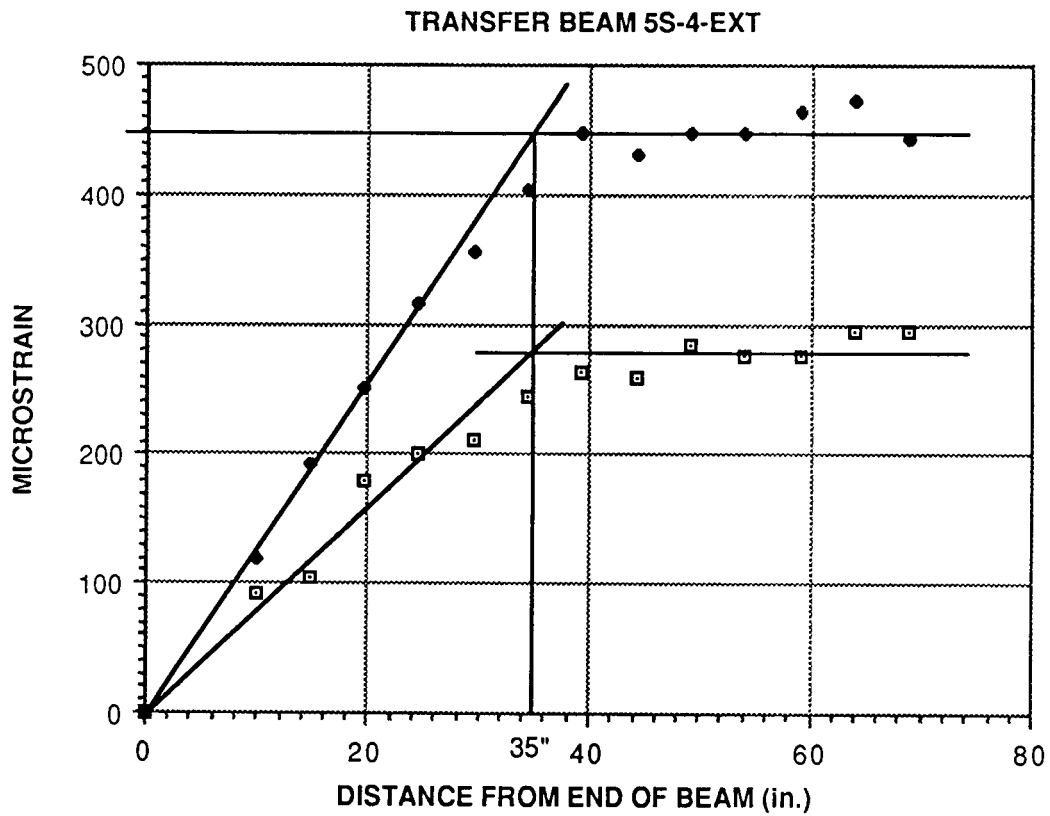


TRANSFER BEAM 5S-1-INT

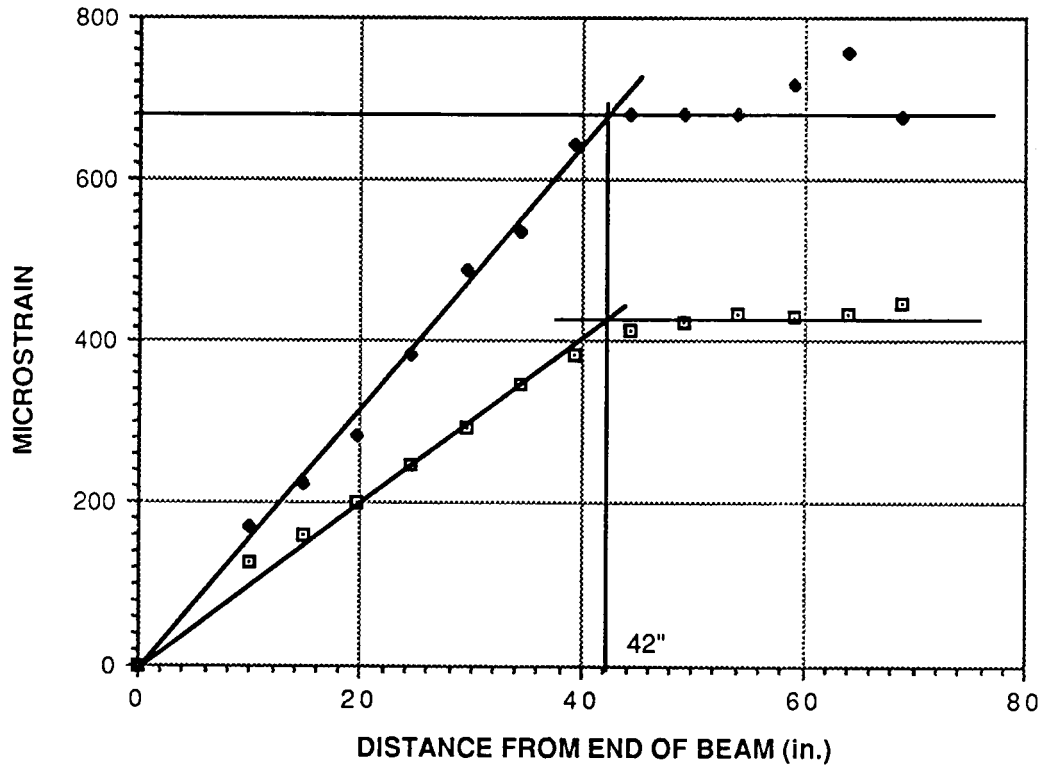




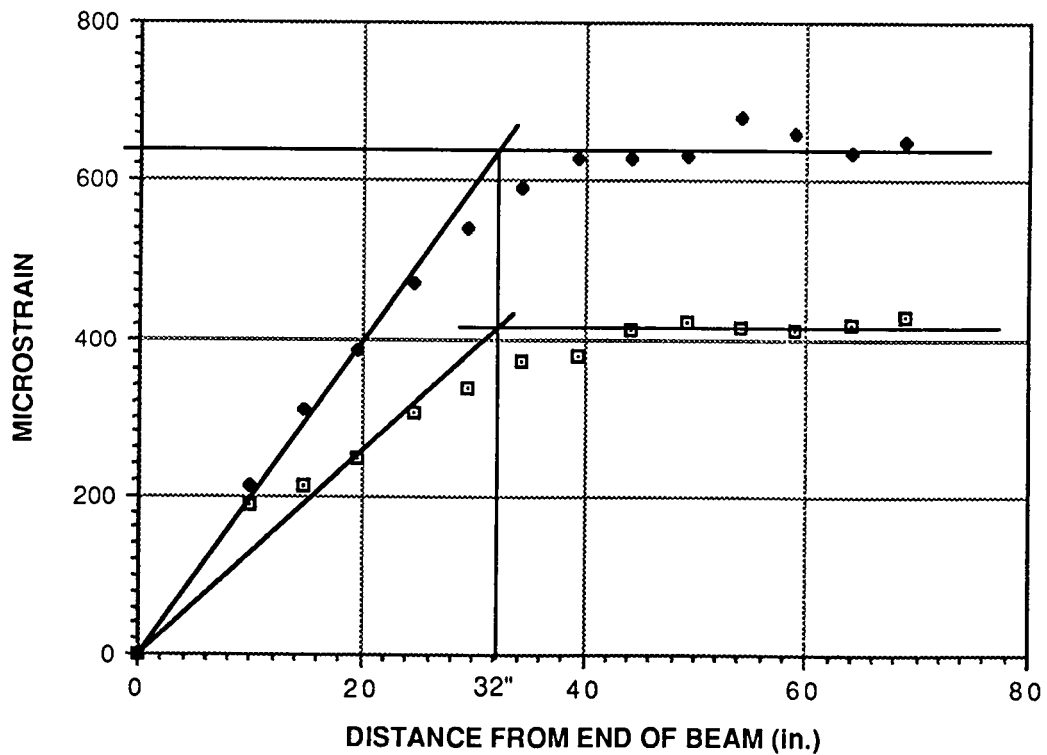




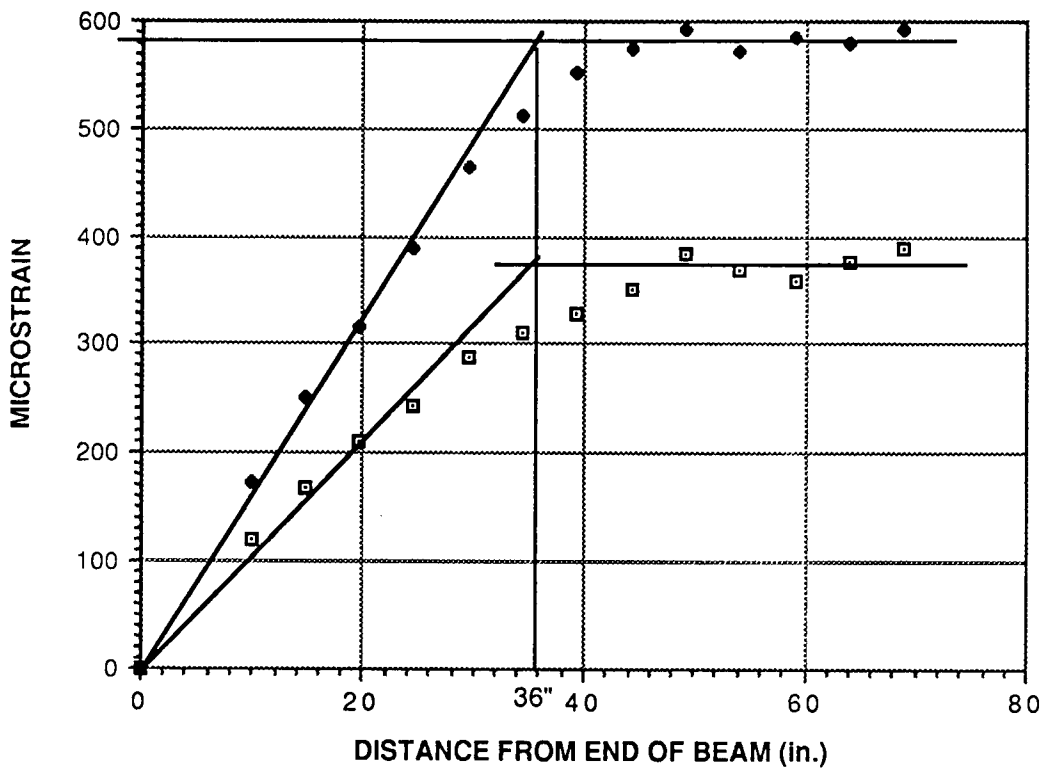
TRANSFER BEAM 916-1-EXT



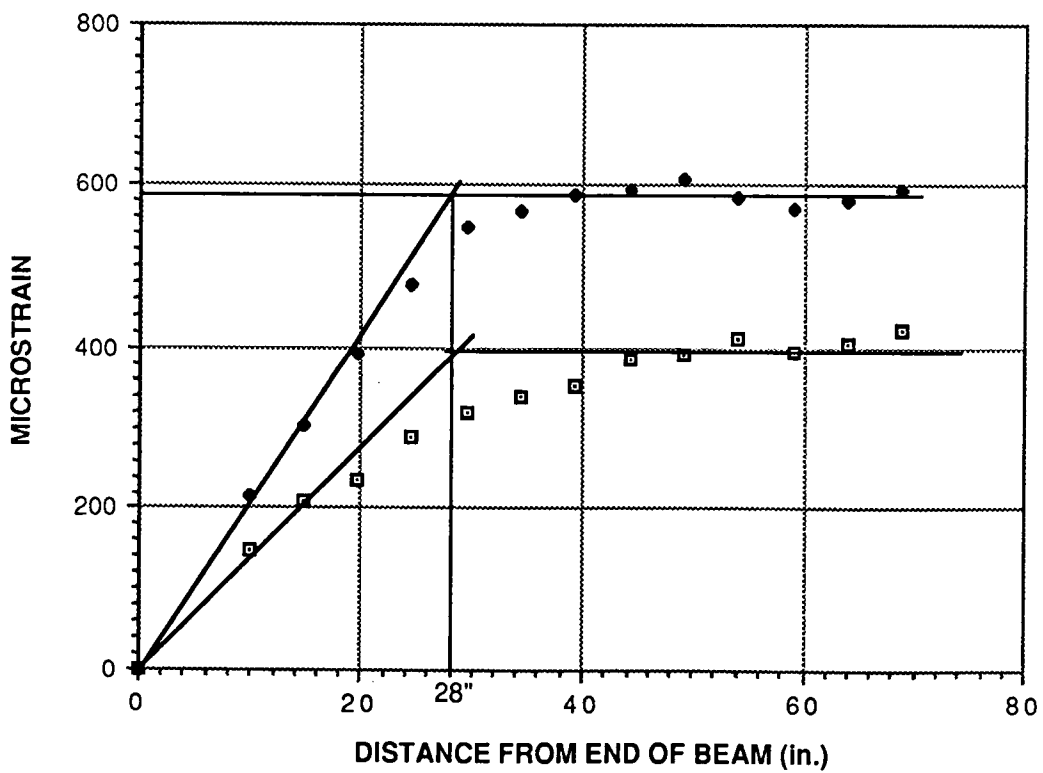
TRANSFER BEAM 916-1-INT

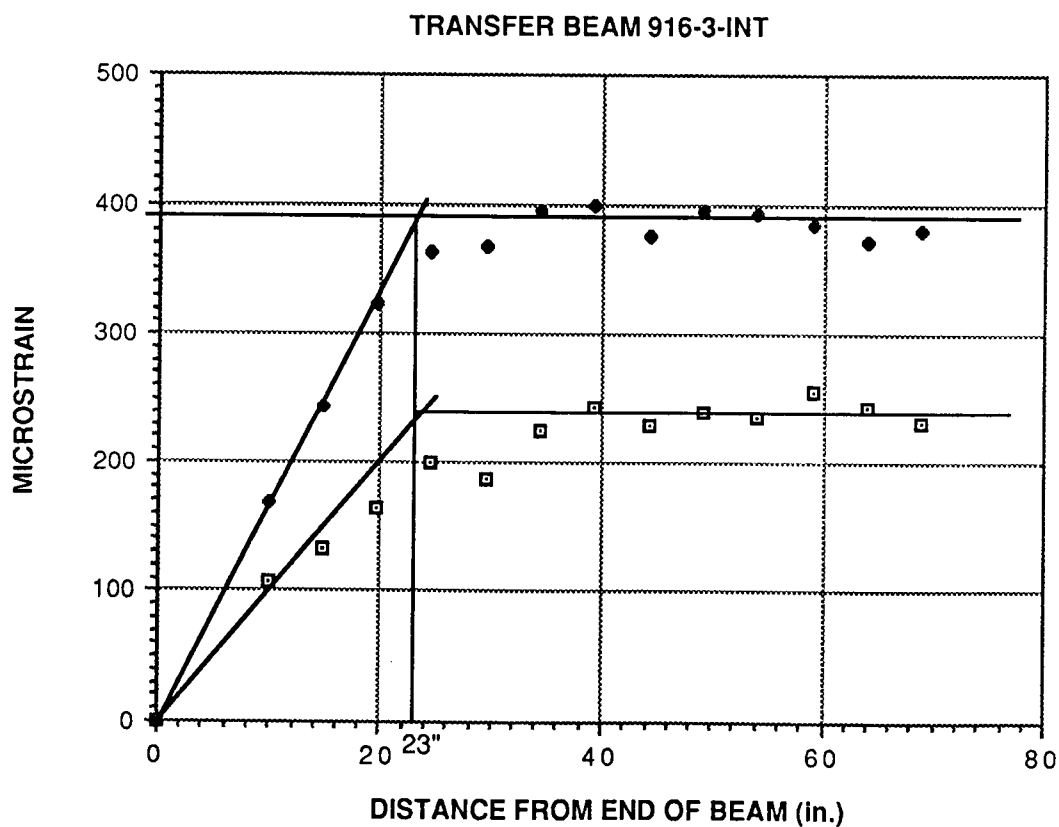
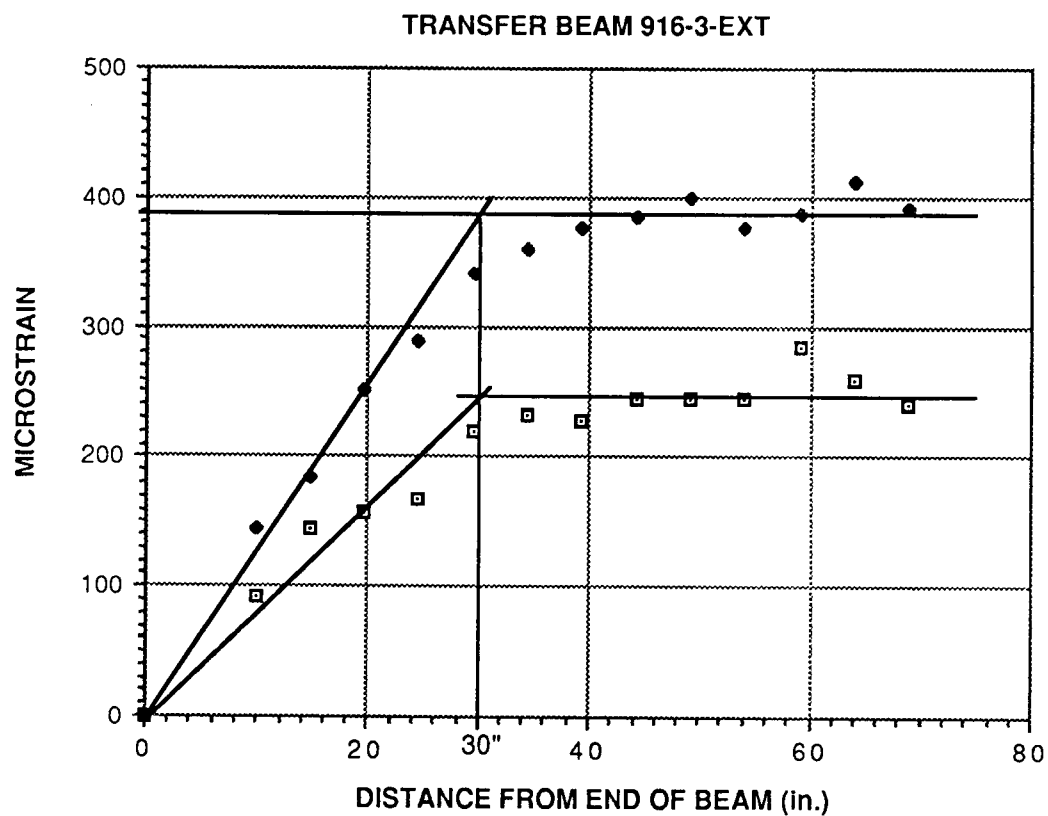


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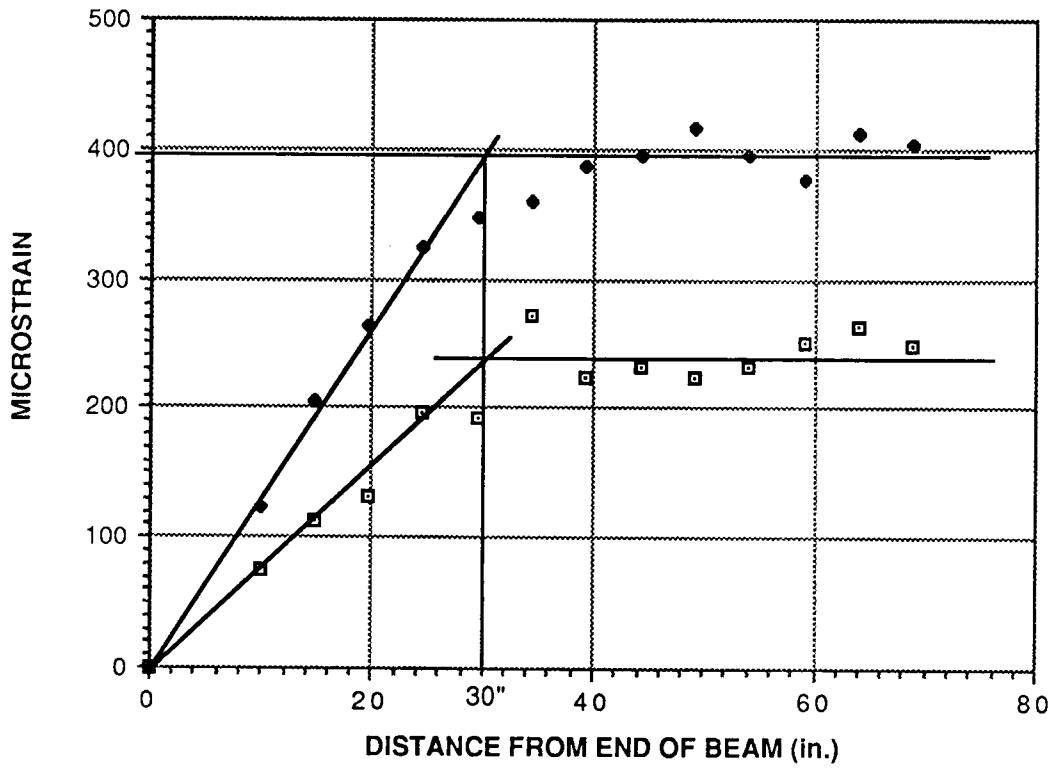


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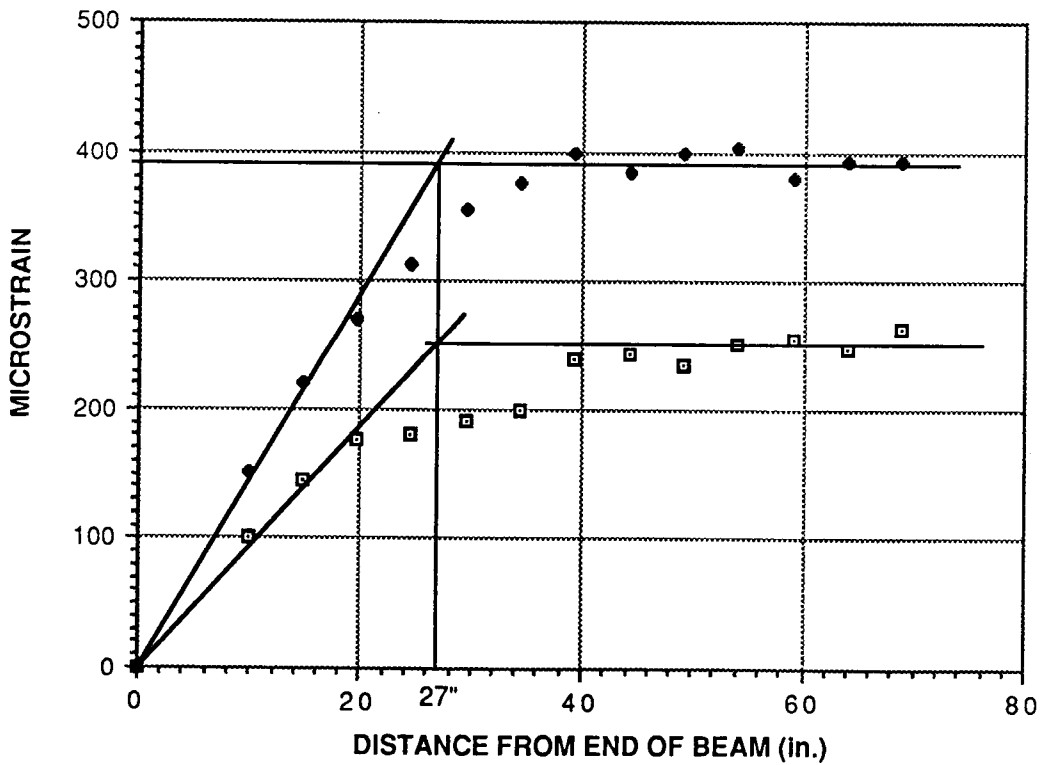


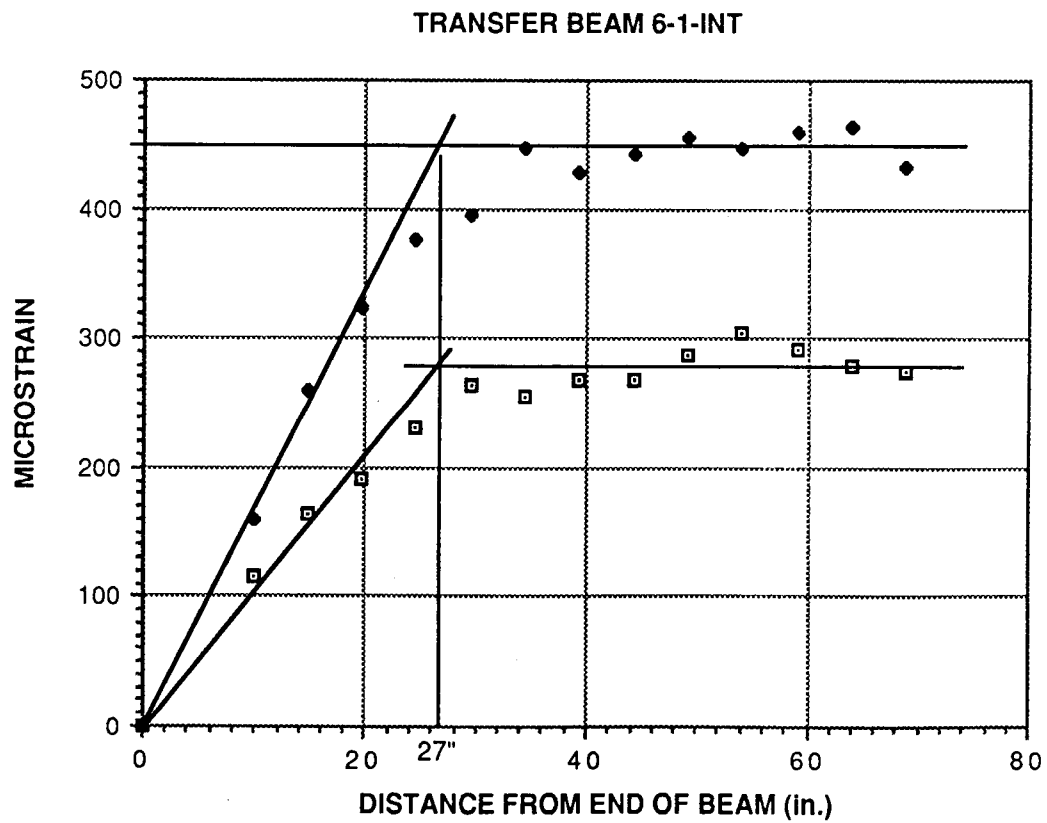
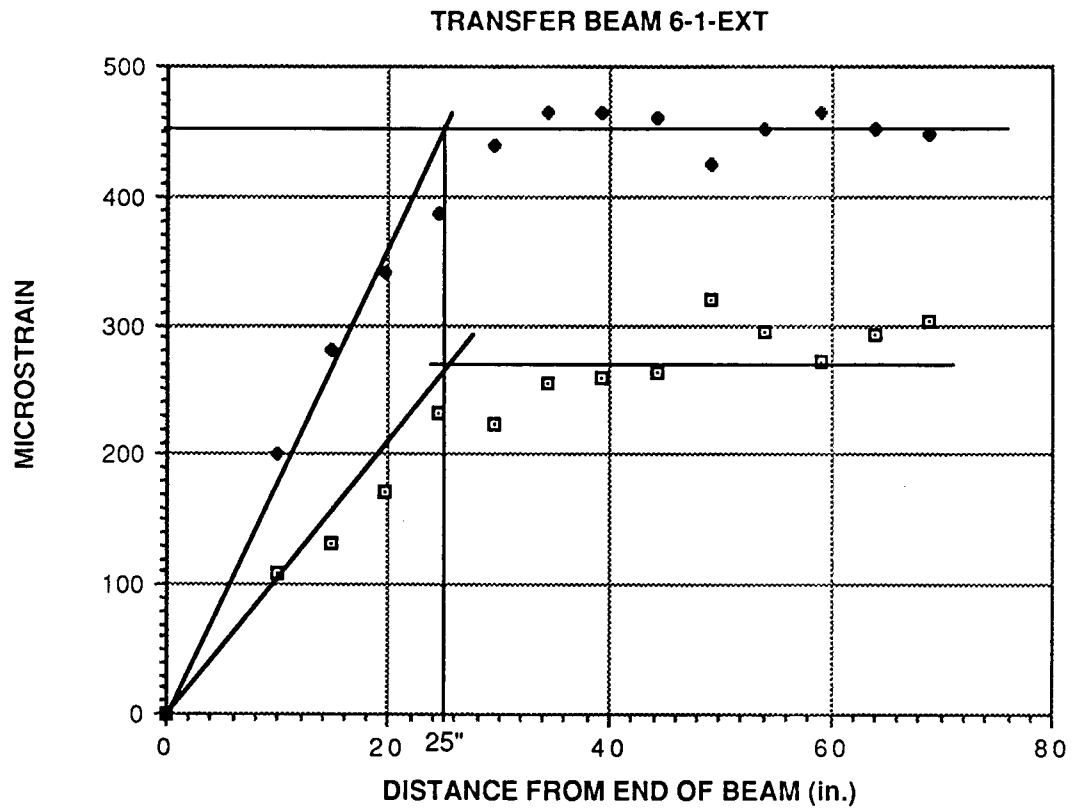


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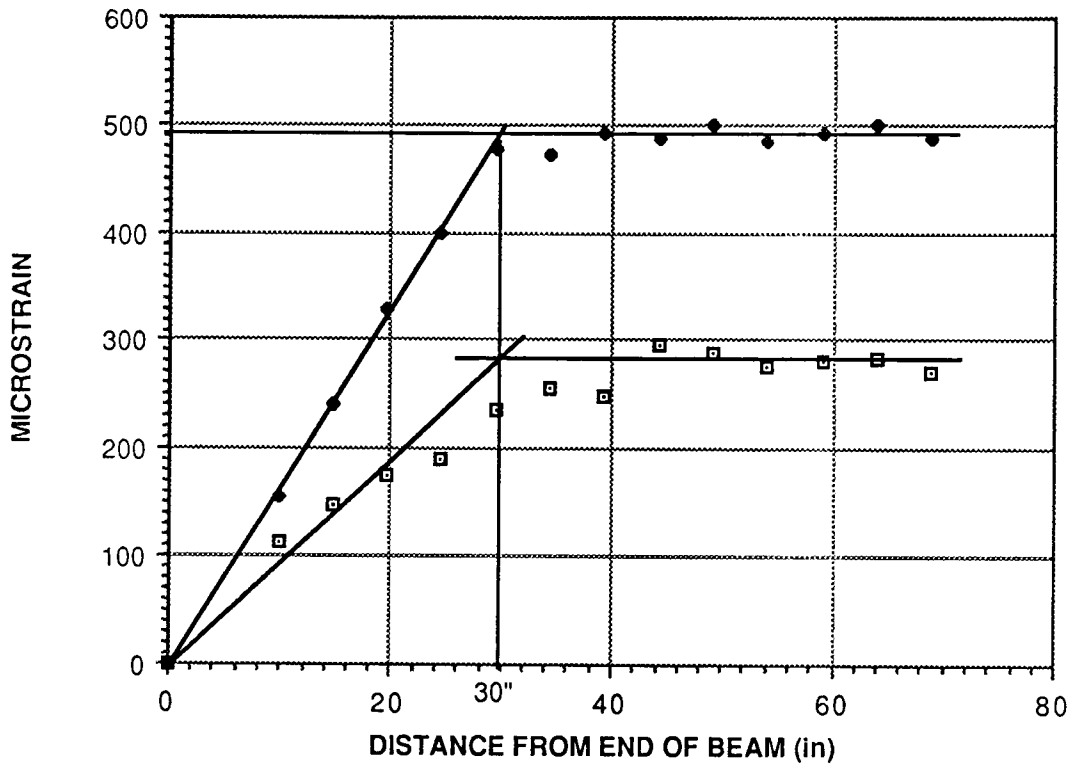


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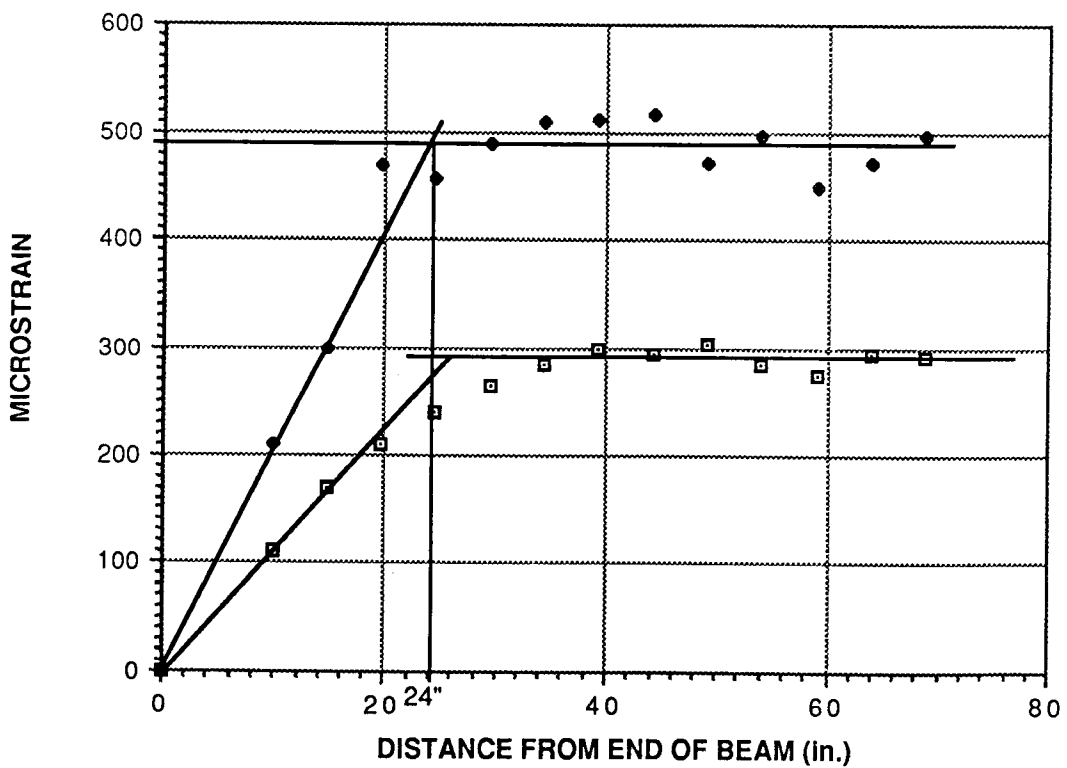


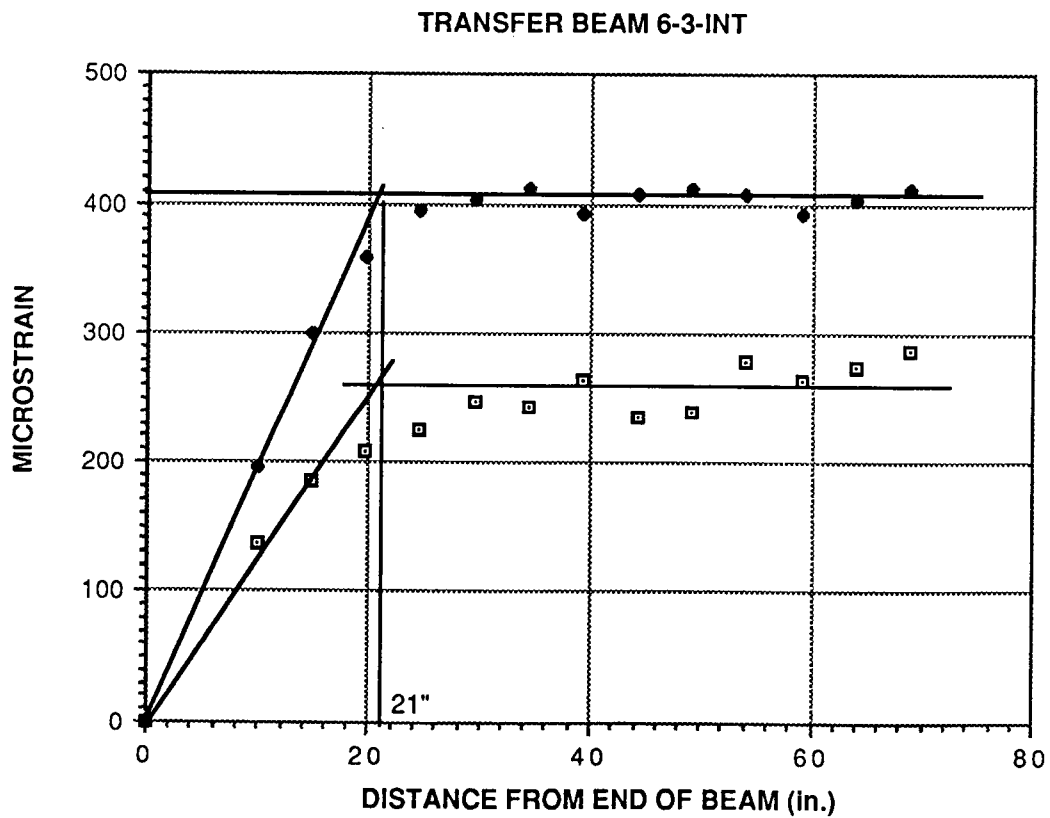
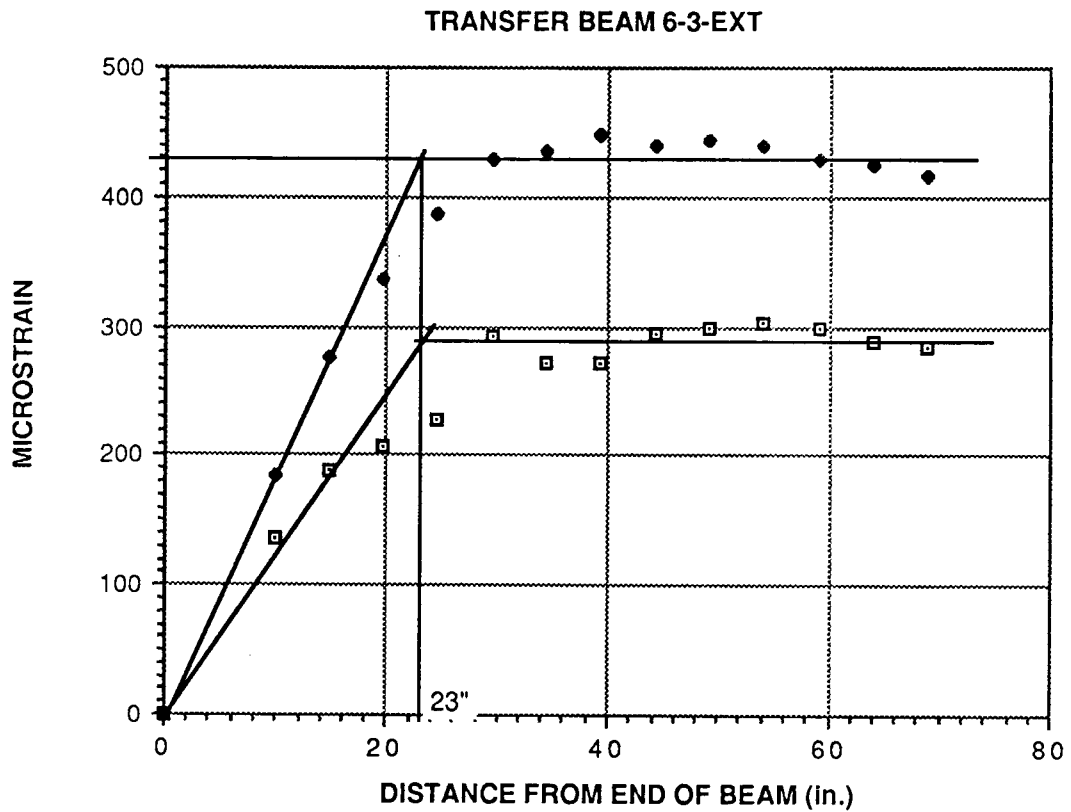


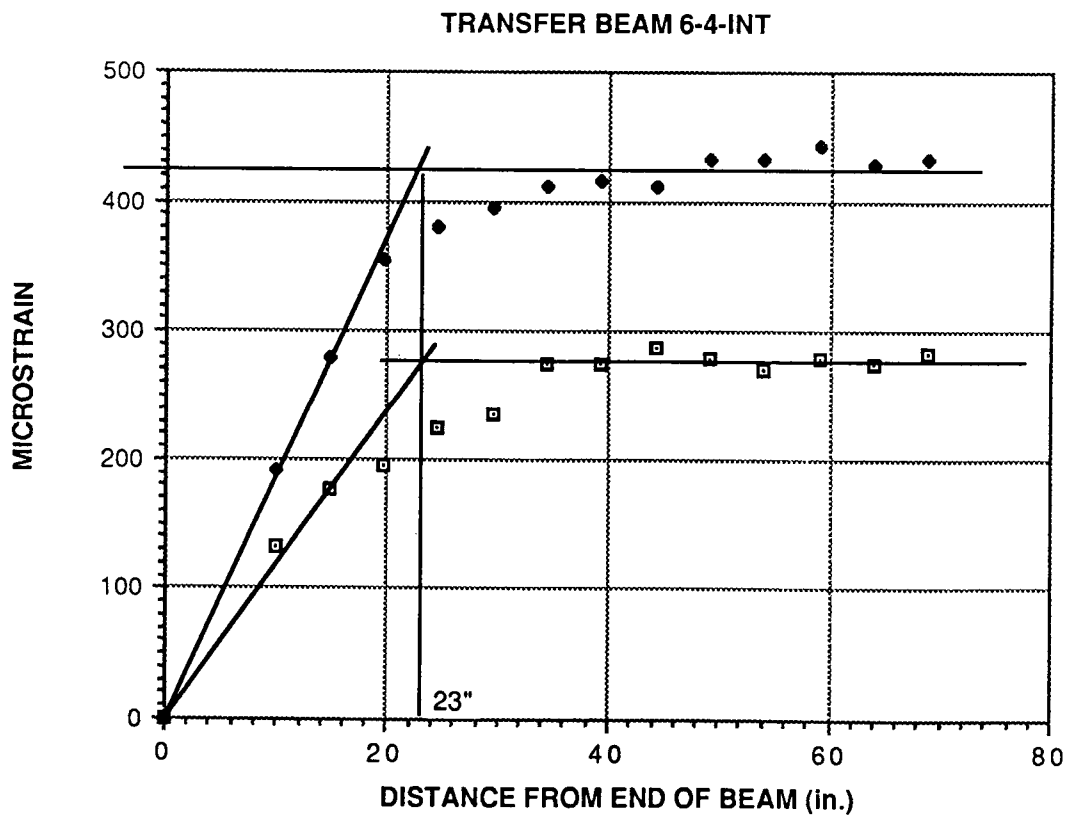
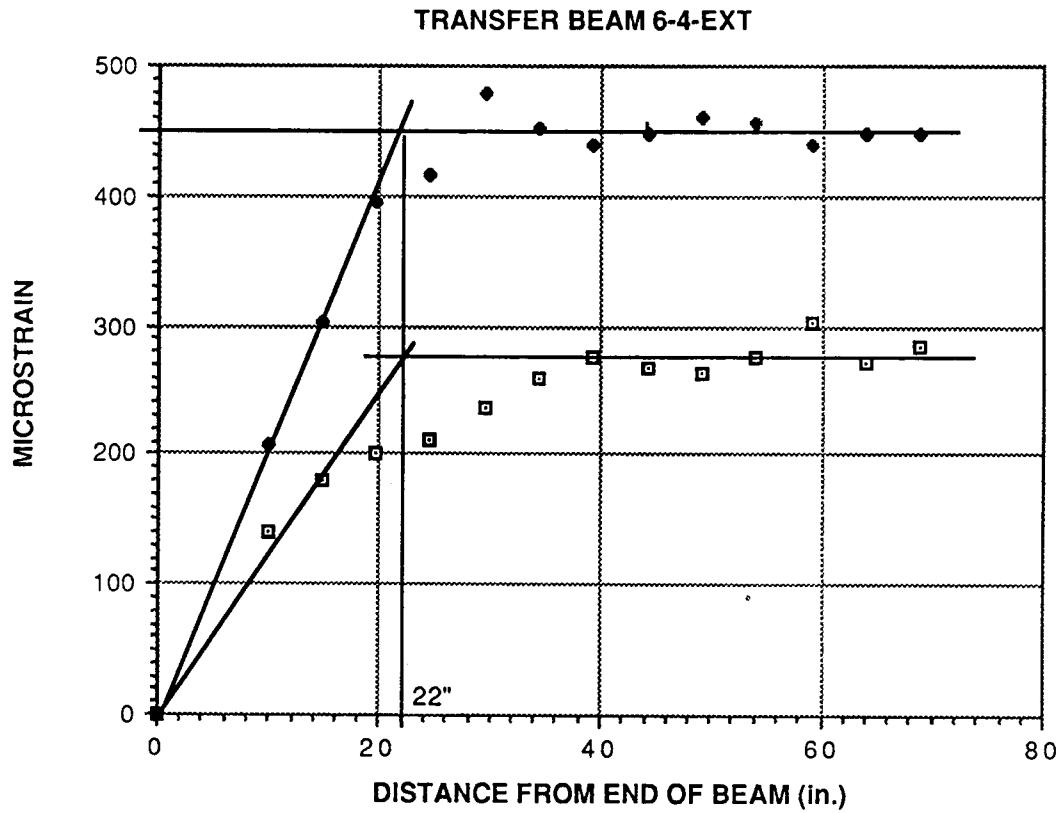
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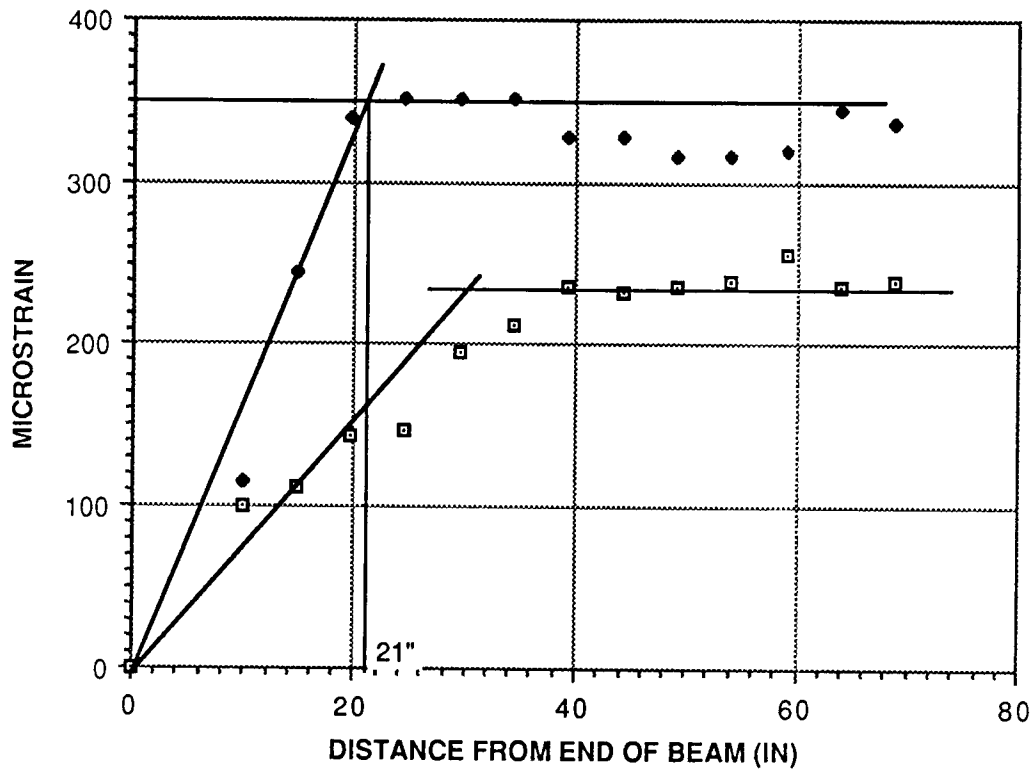
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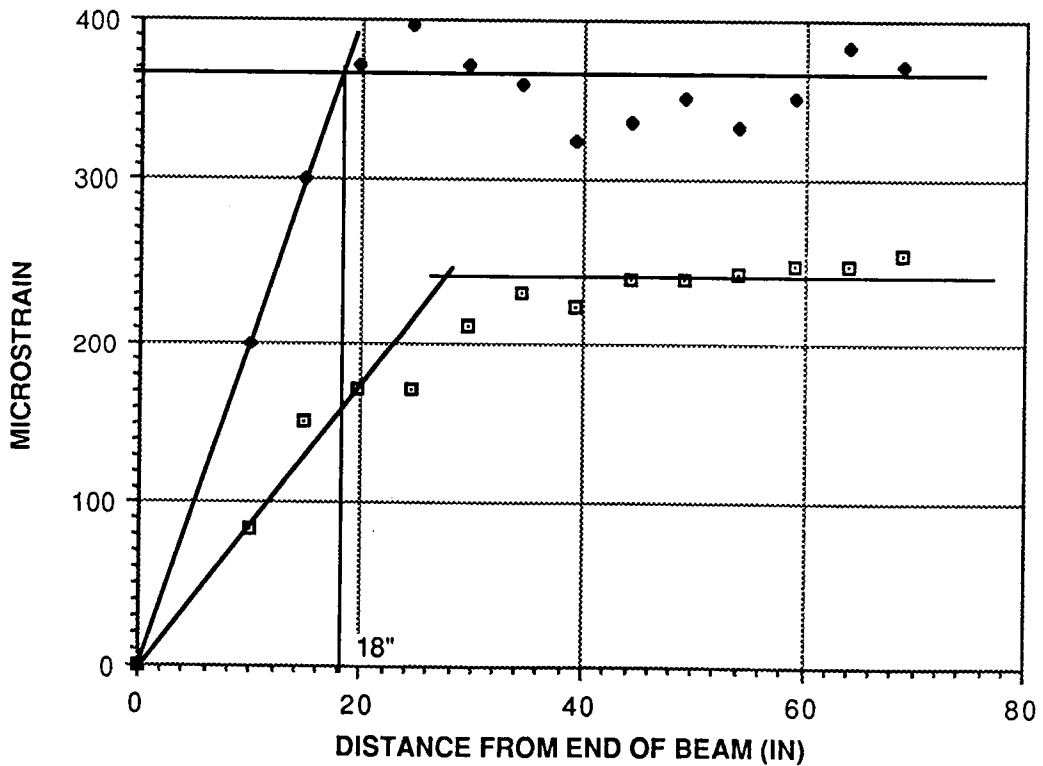


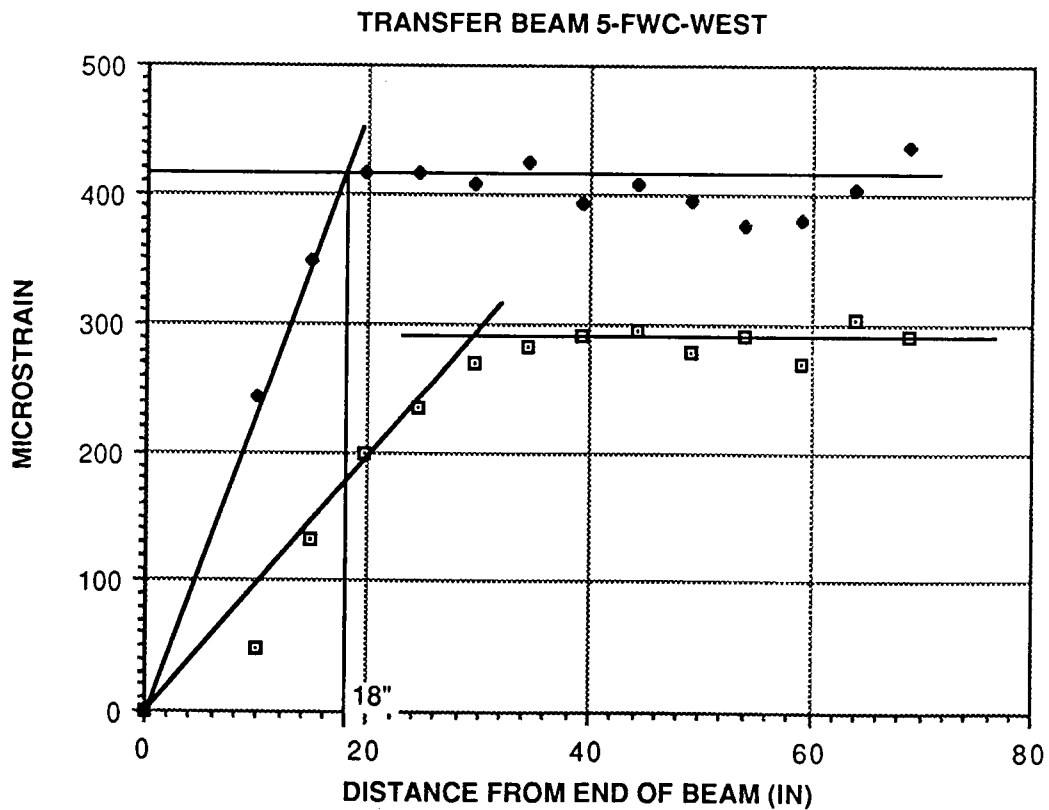
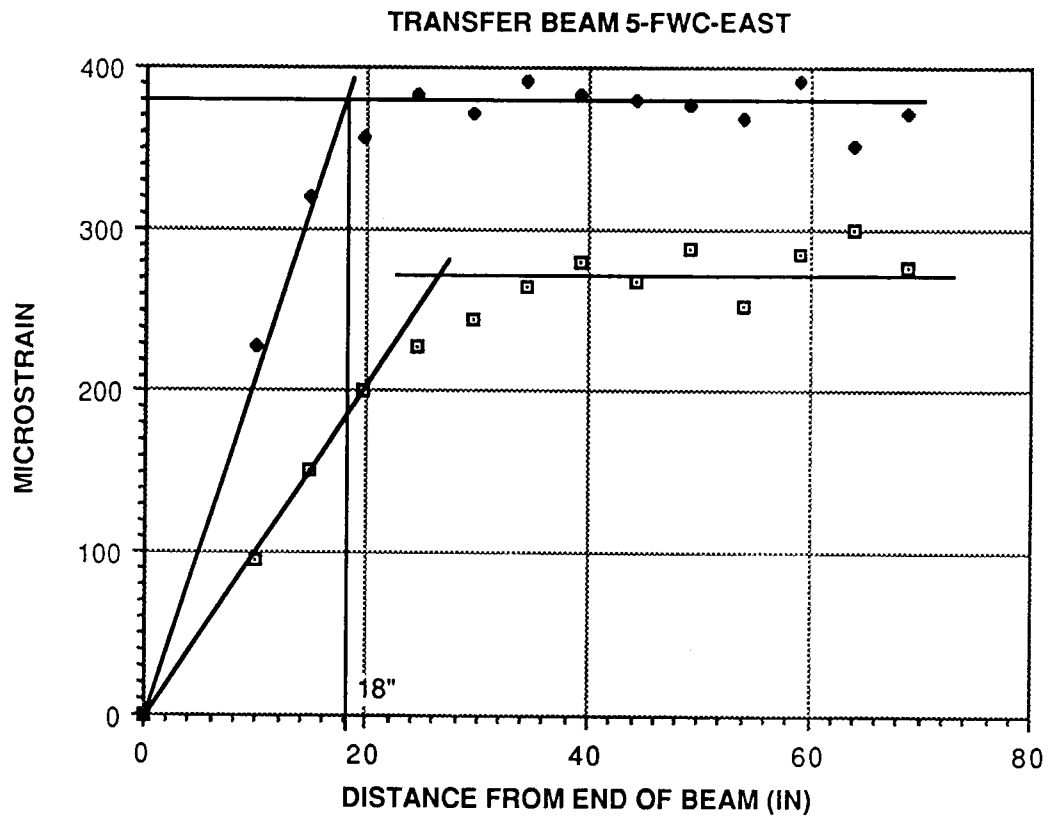


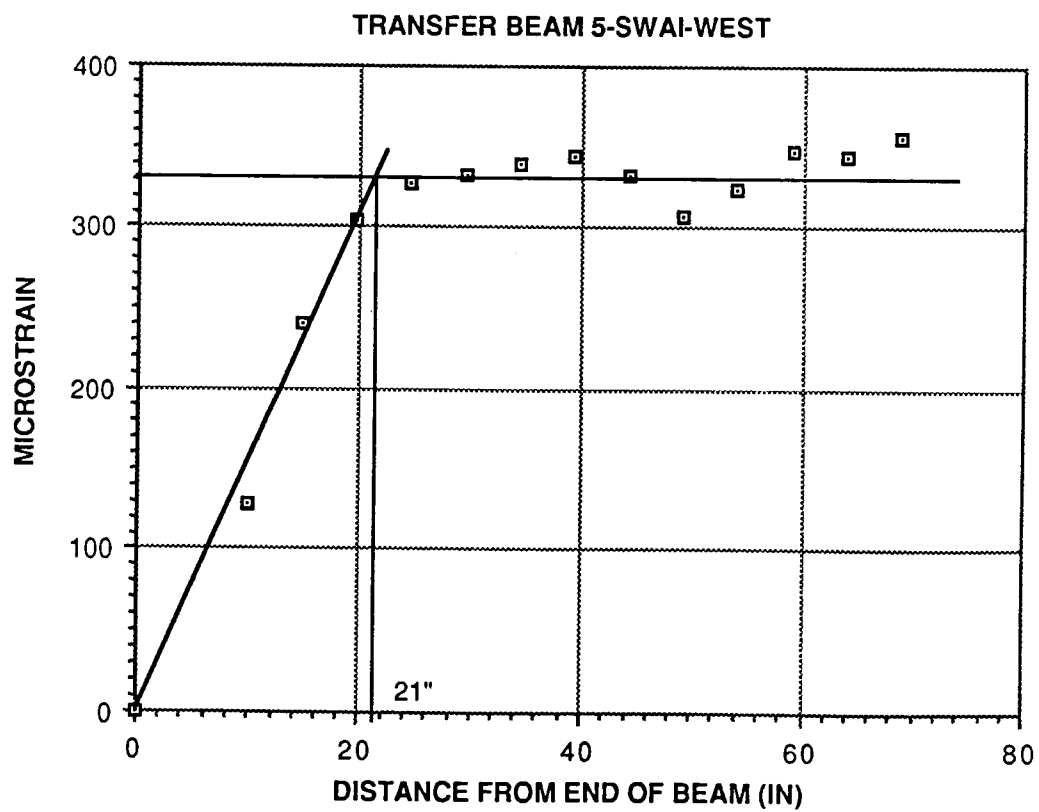
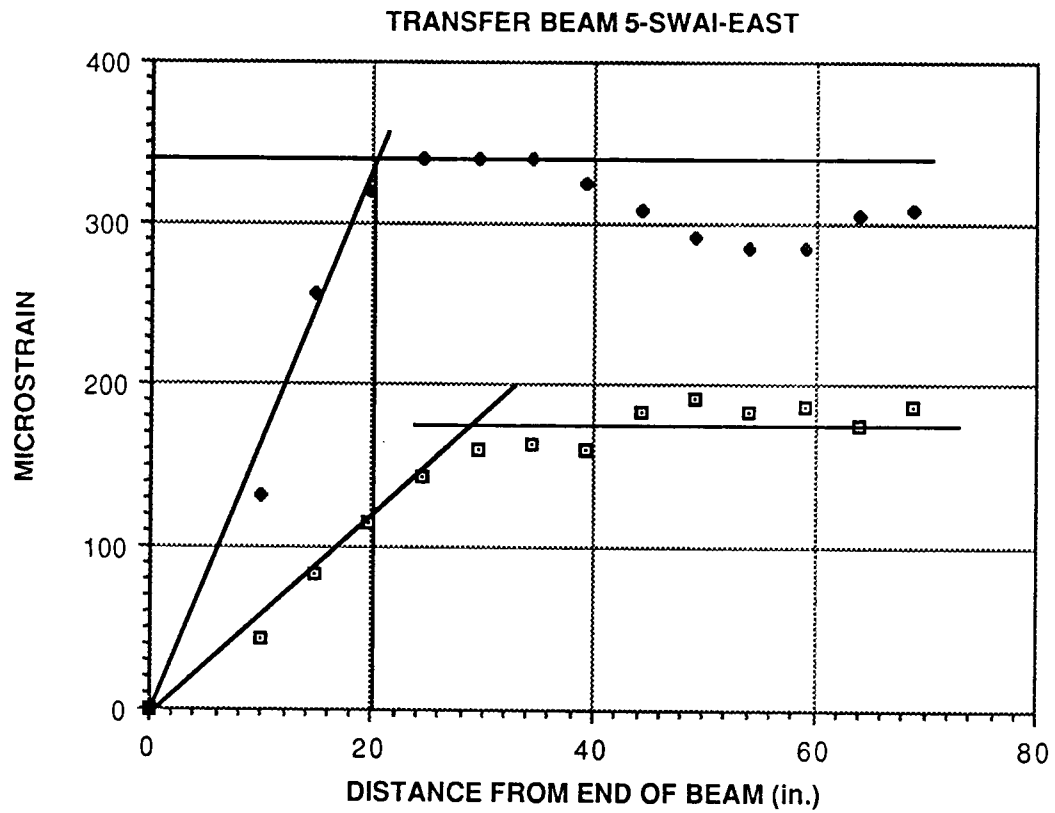
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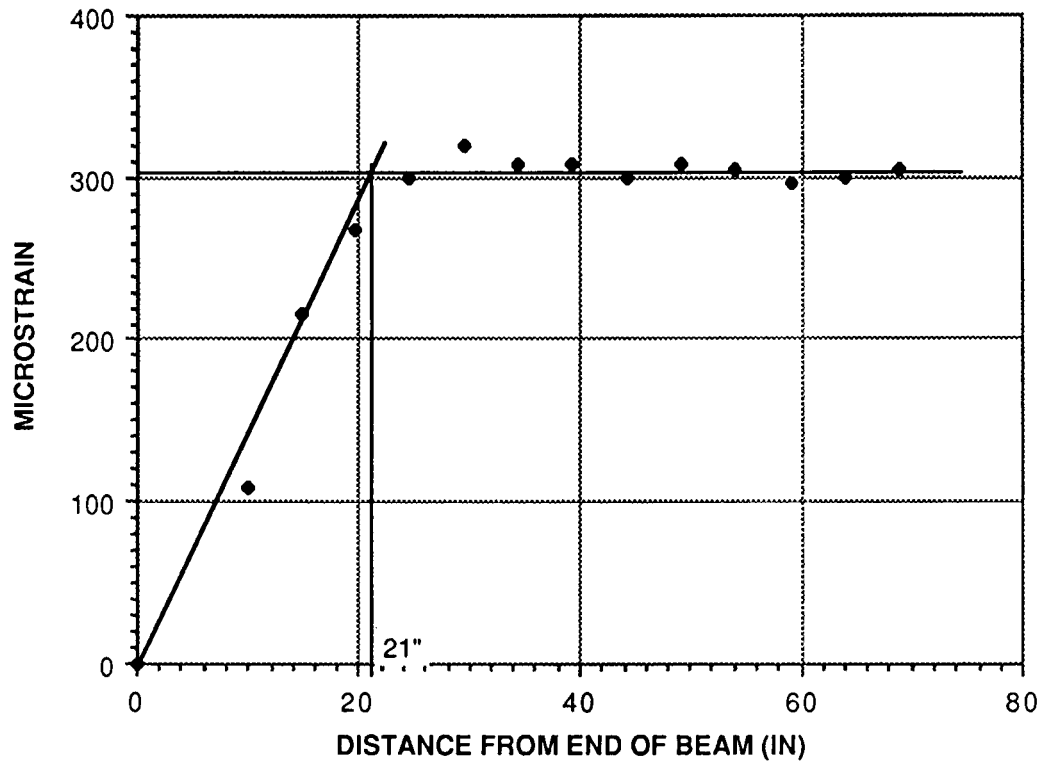
TRANSFER BEAM 5-ASW-WEST



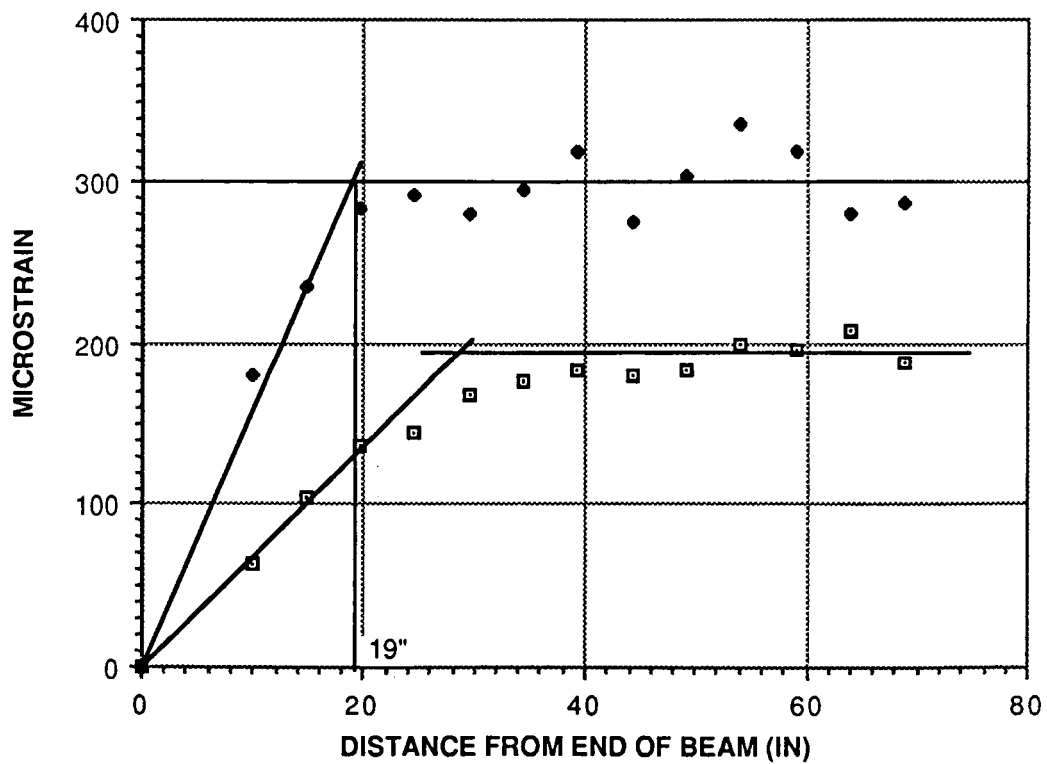


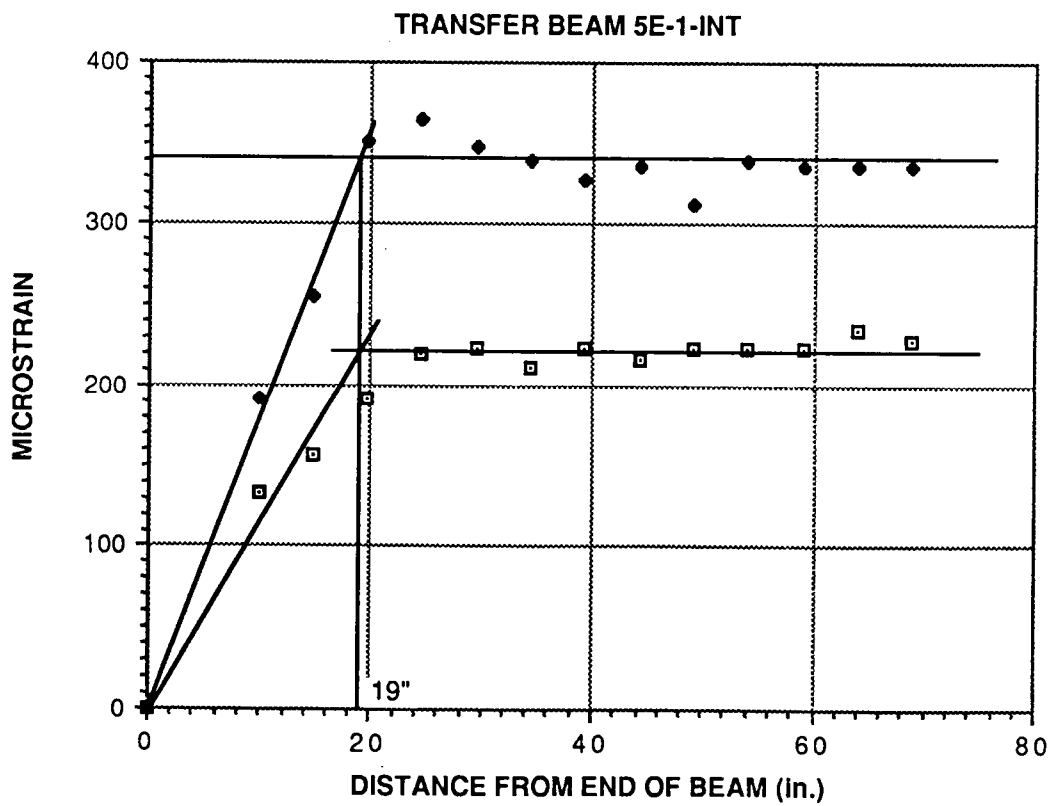
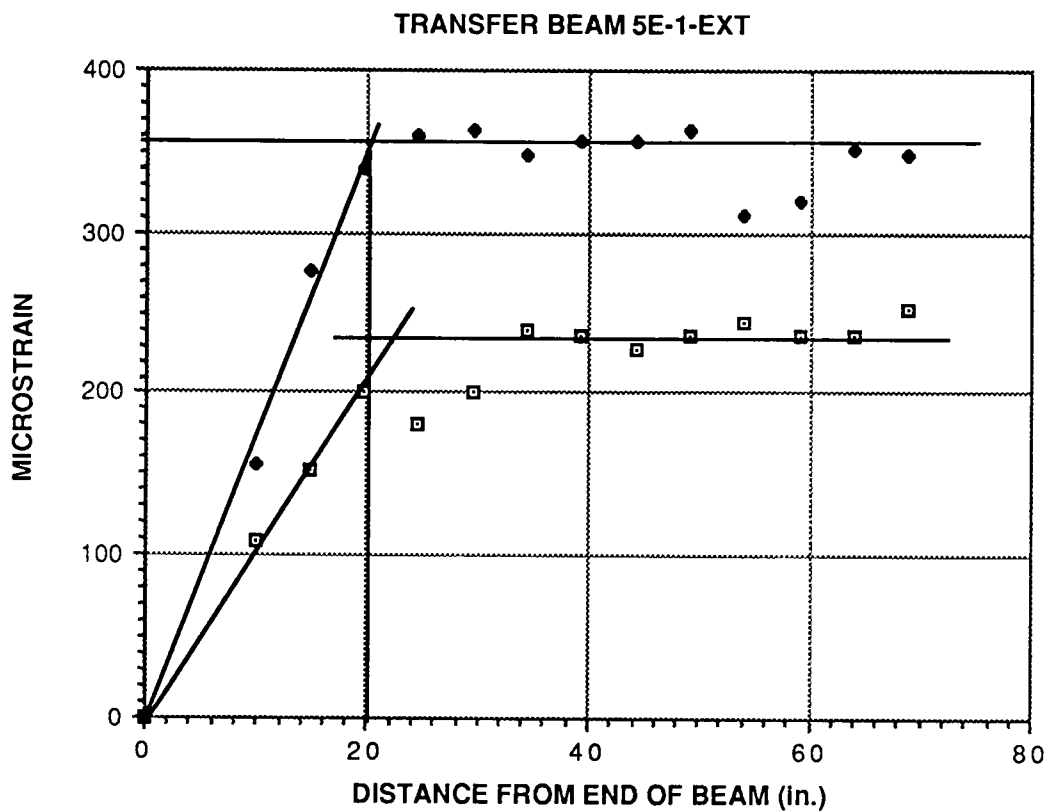


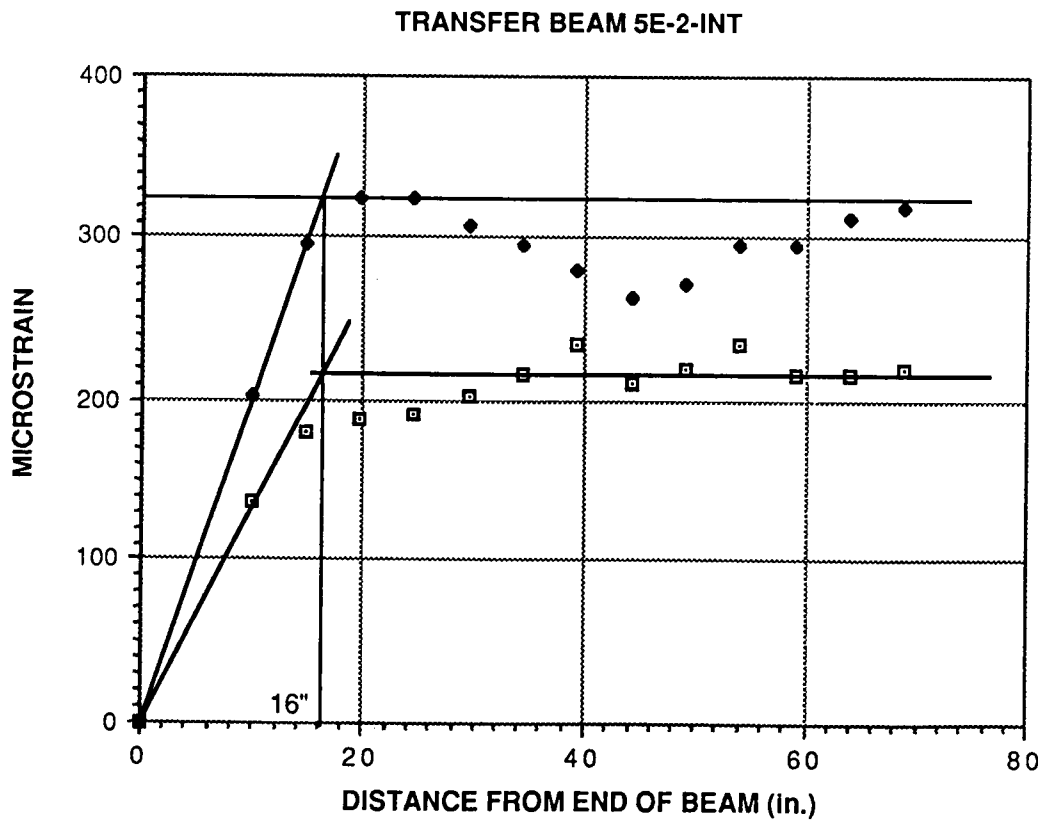
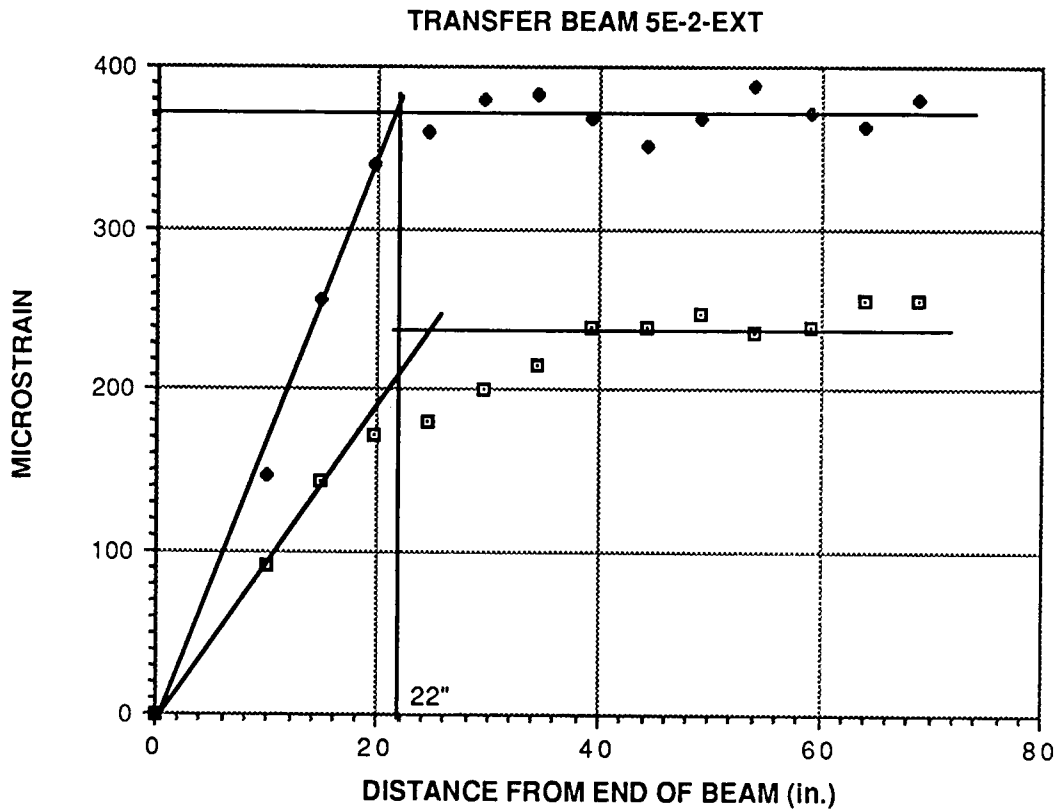
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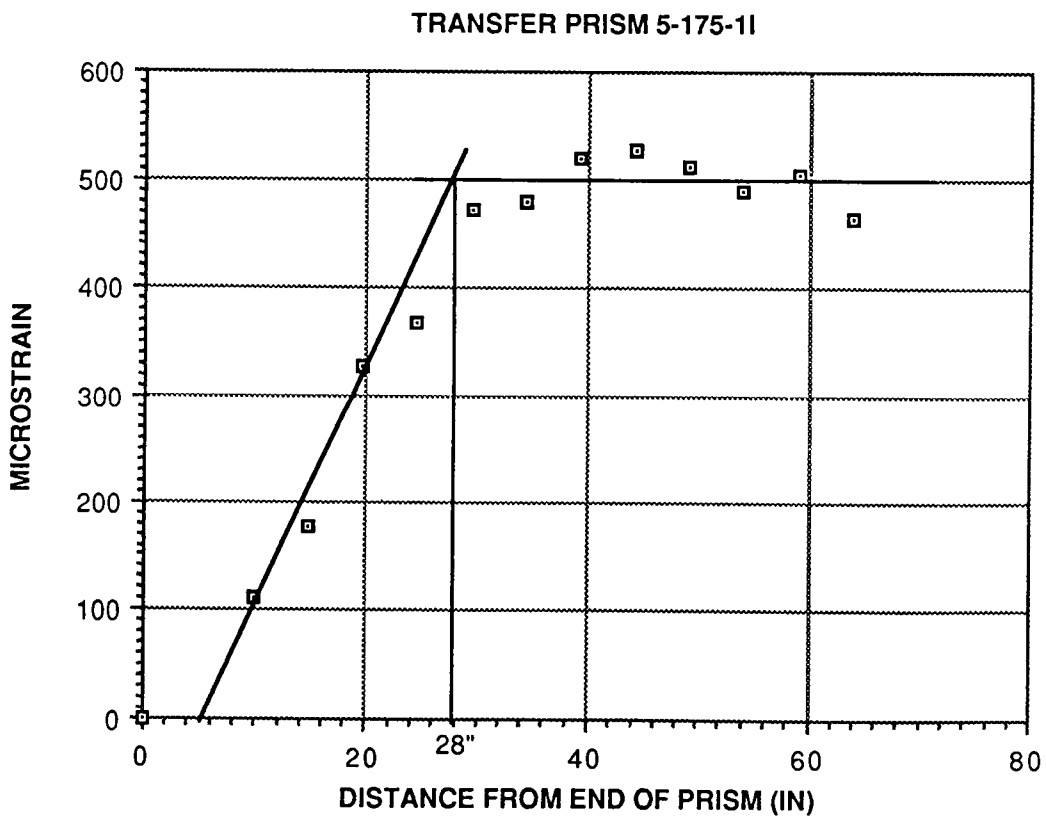
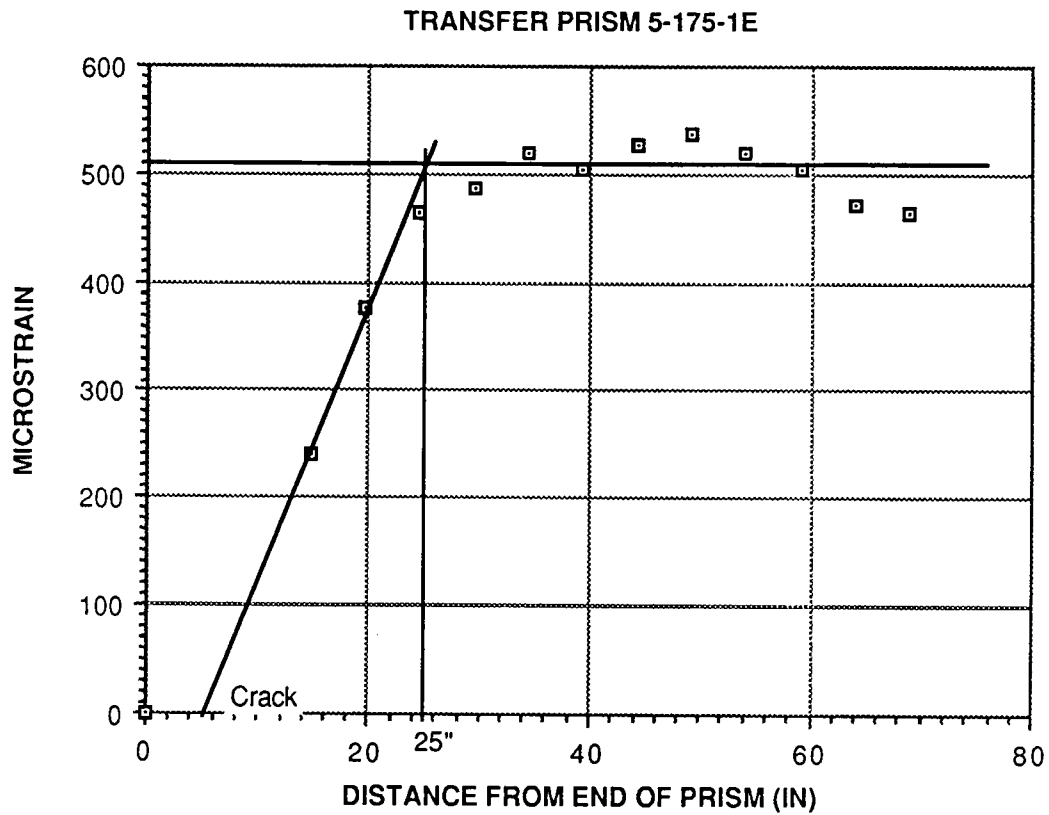


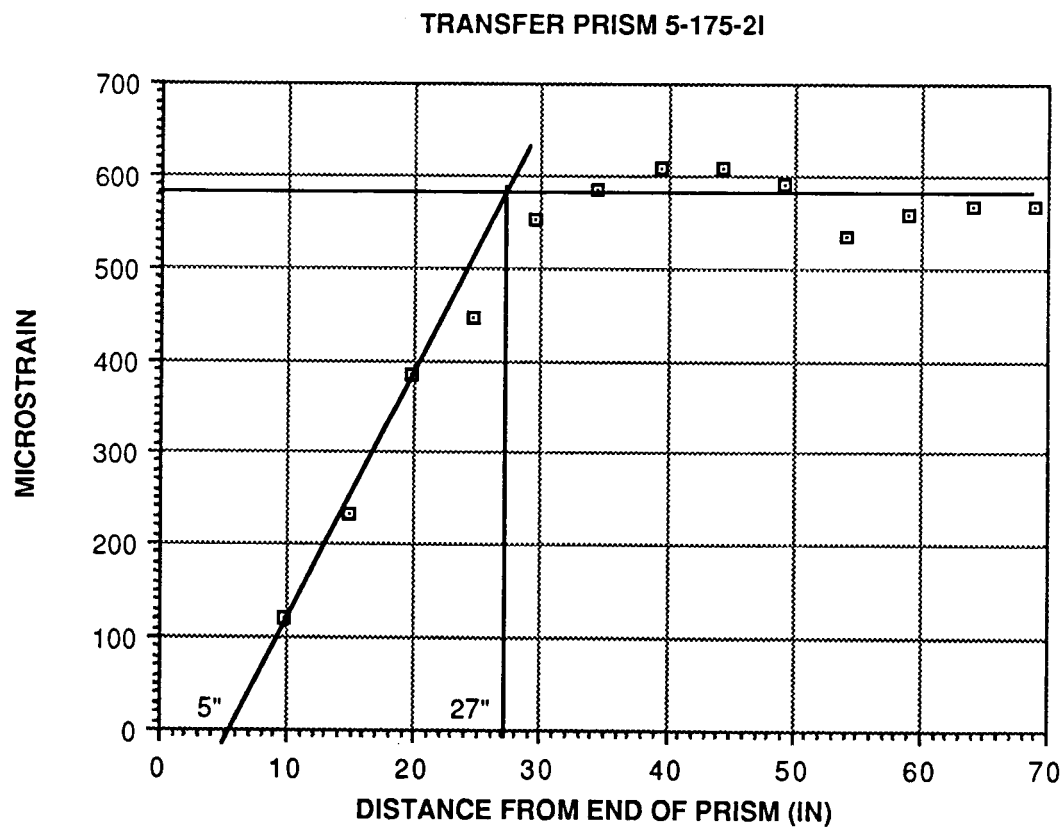
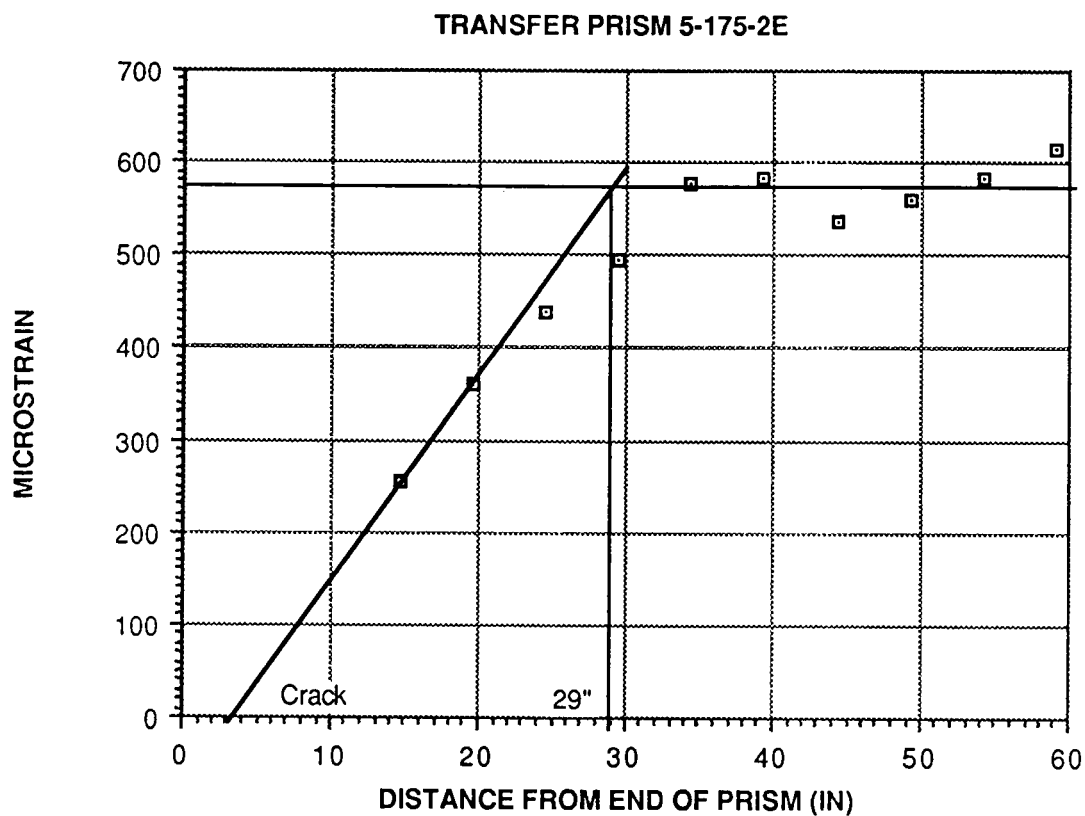
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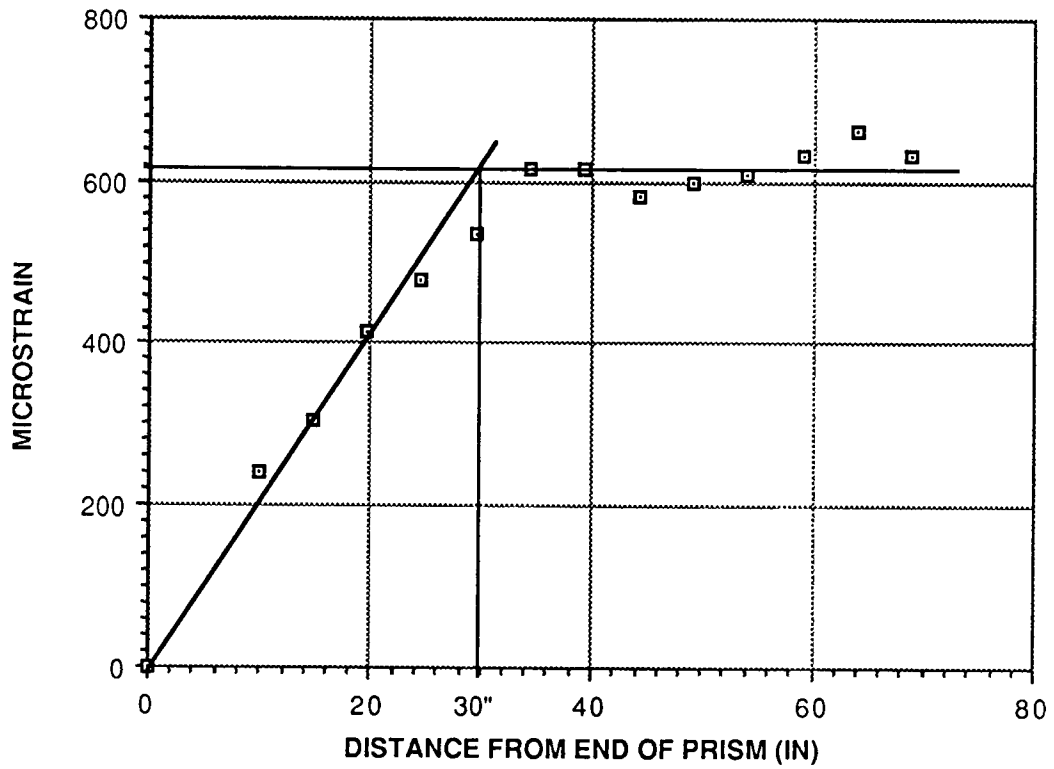




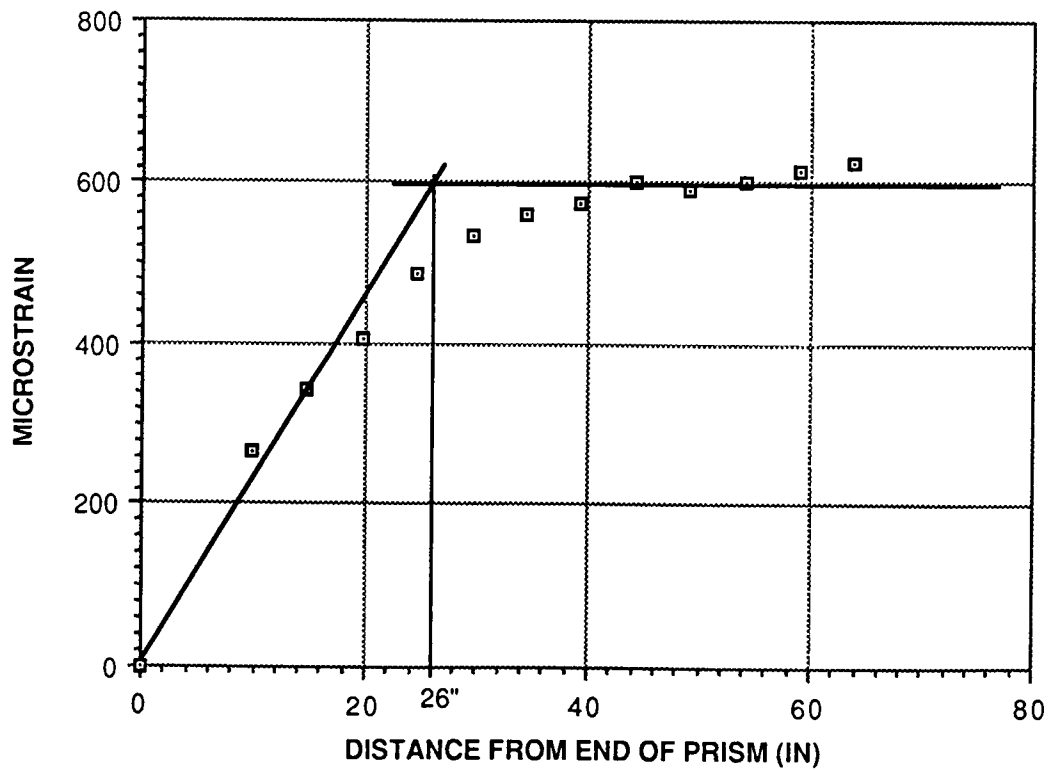


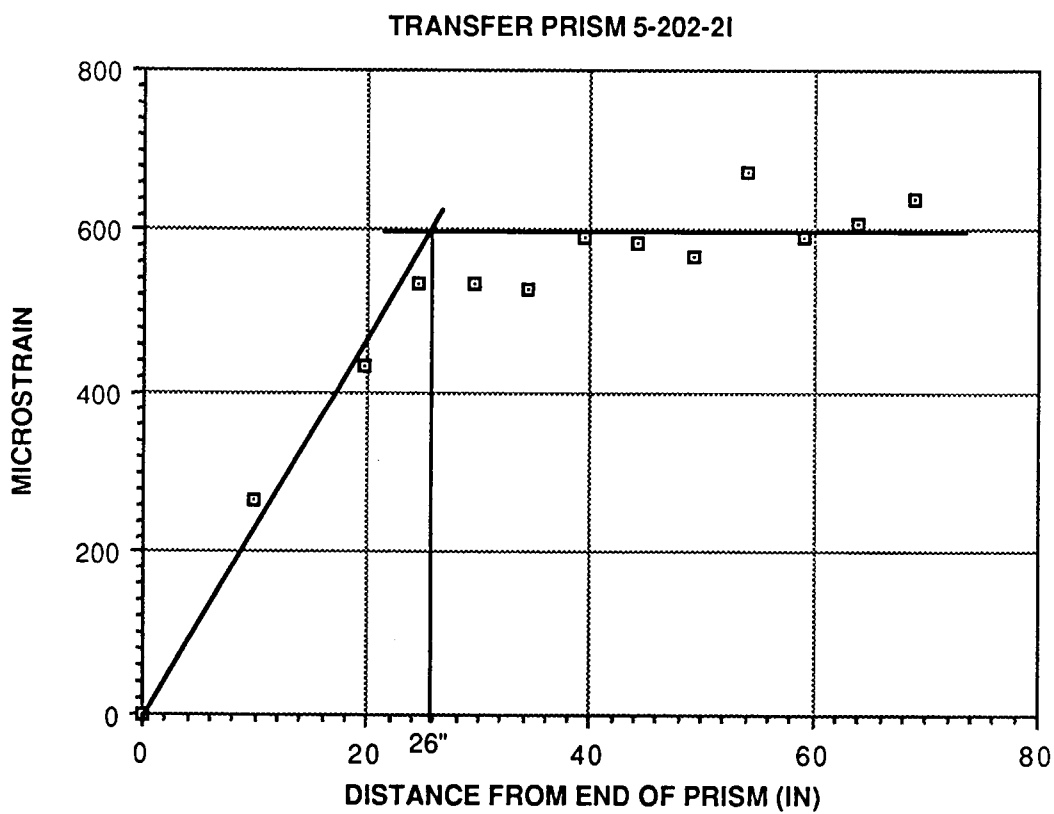
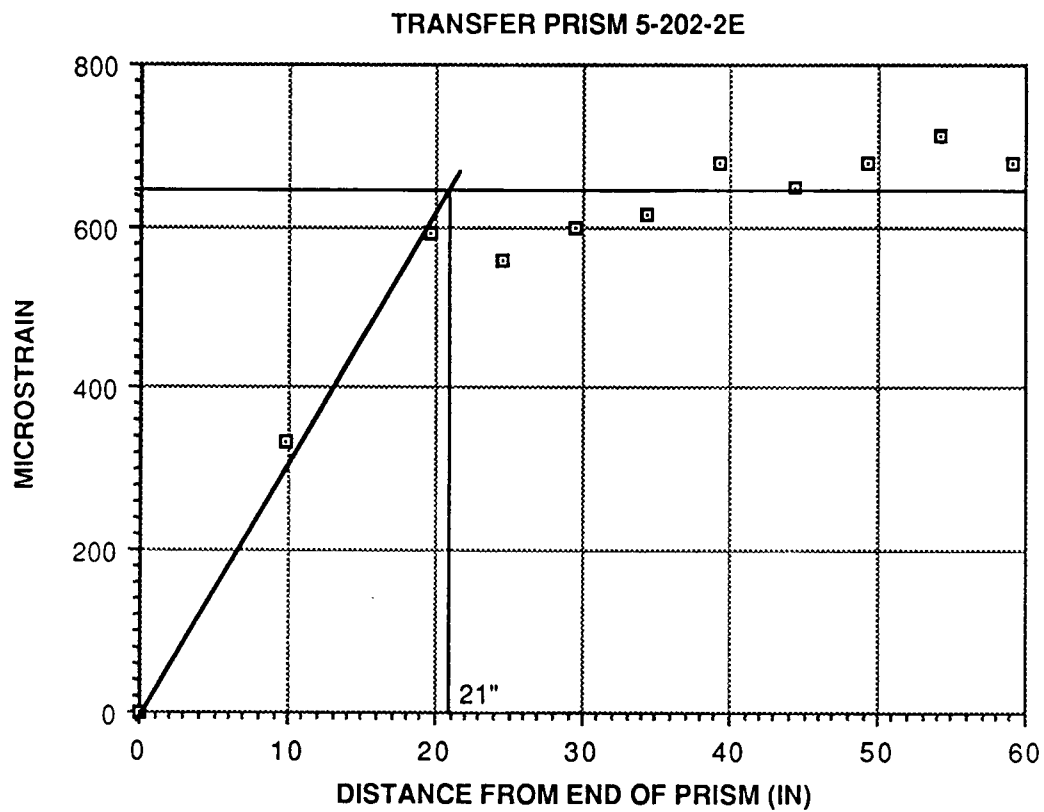


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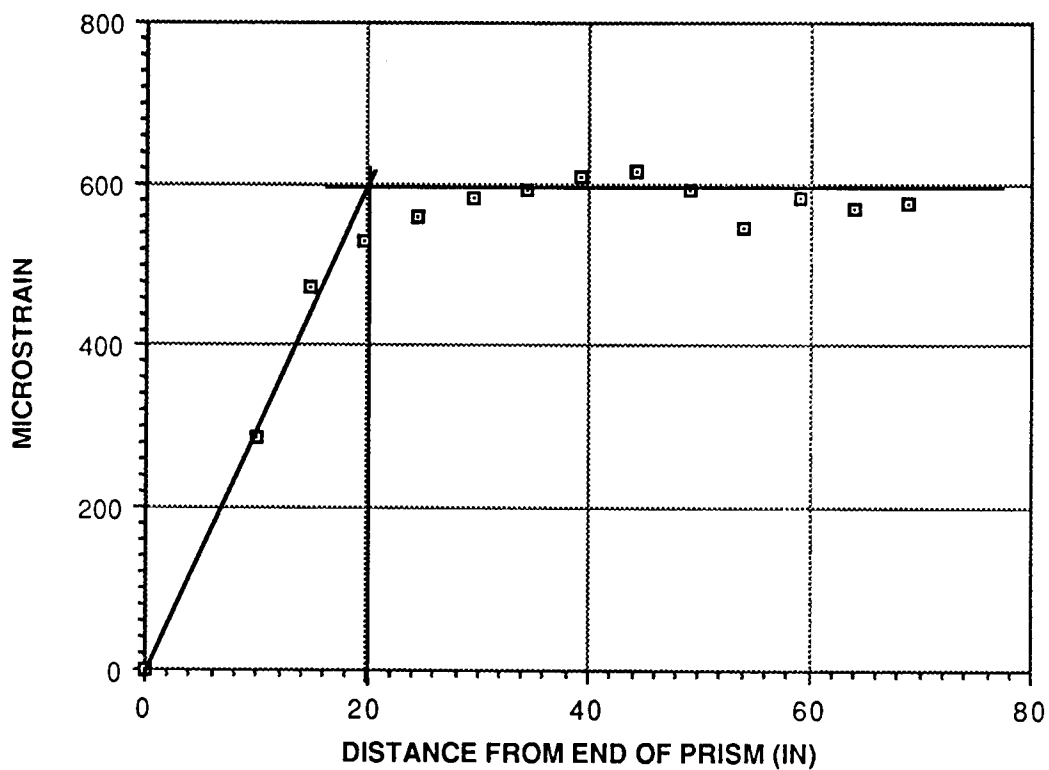


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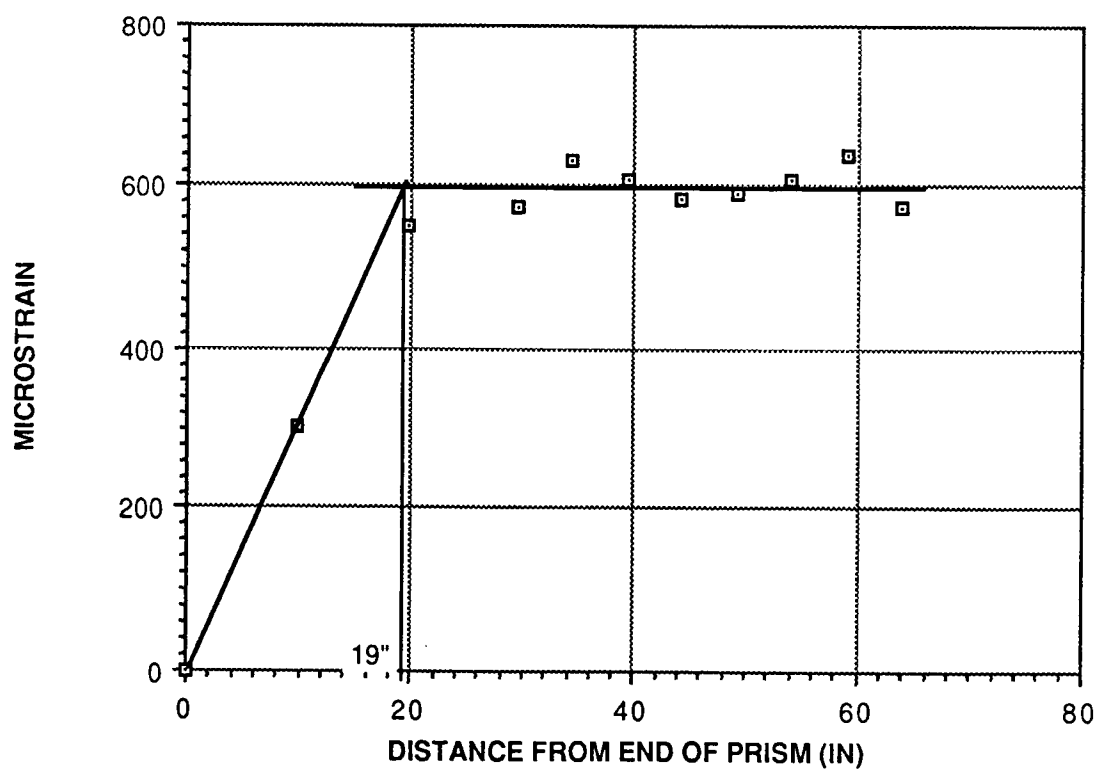




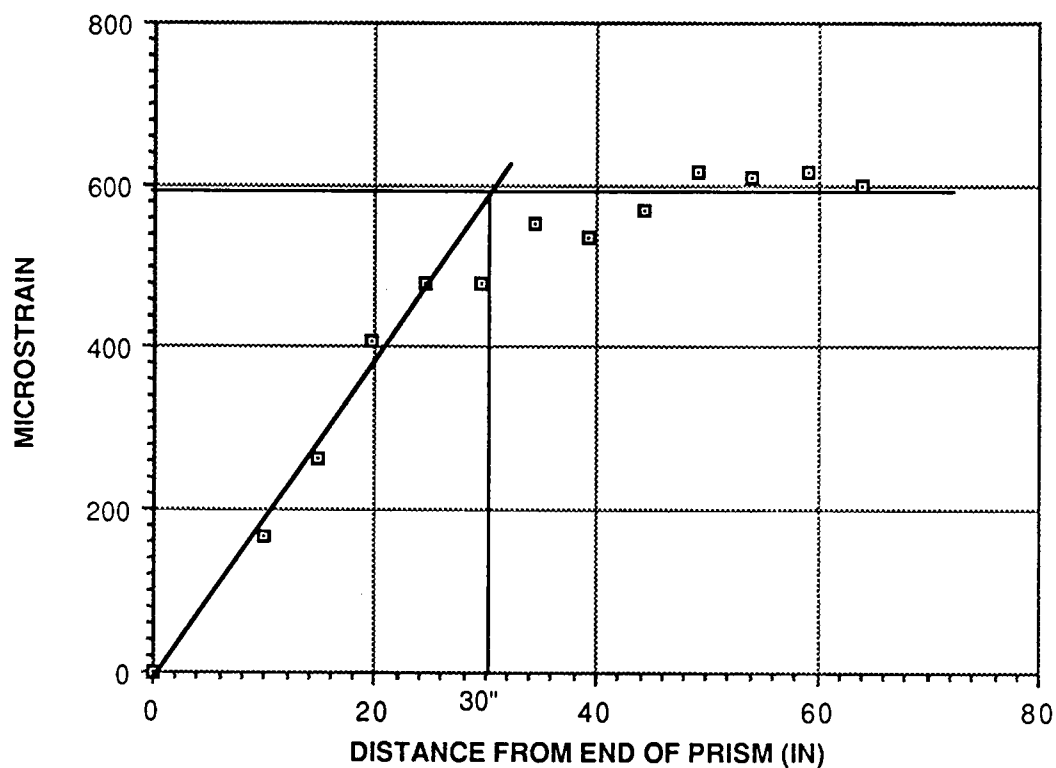
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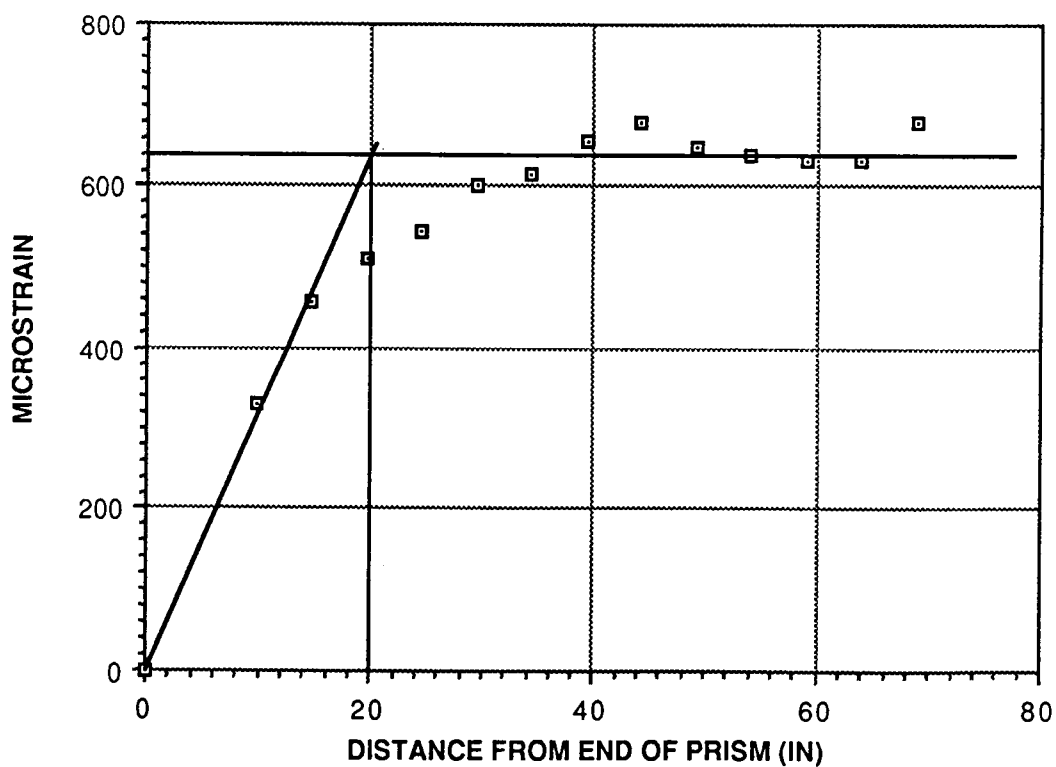
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TRANSFER PRISM 6-202-2E



TRANSFER PRISM 6-202-2I

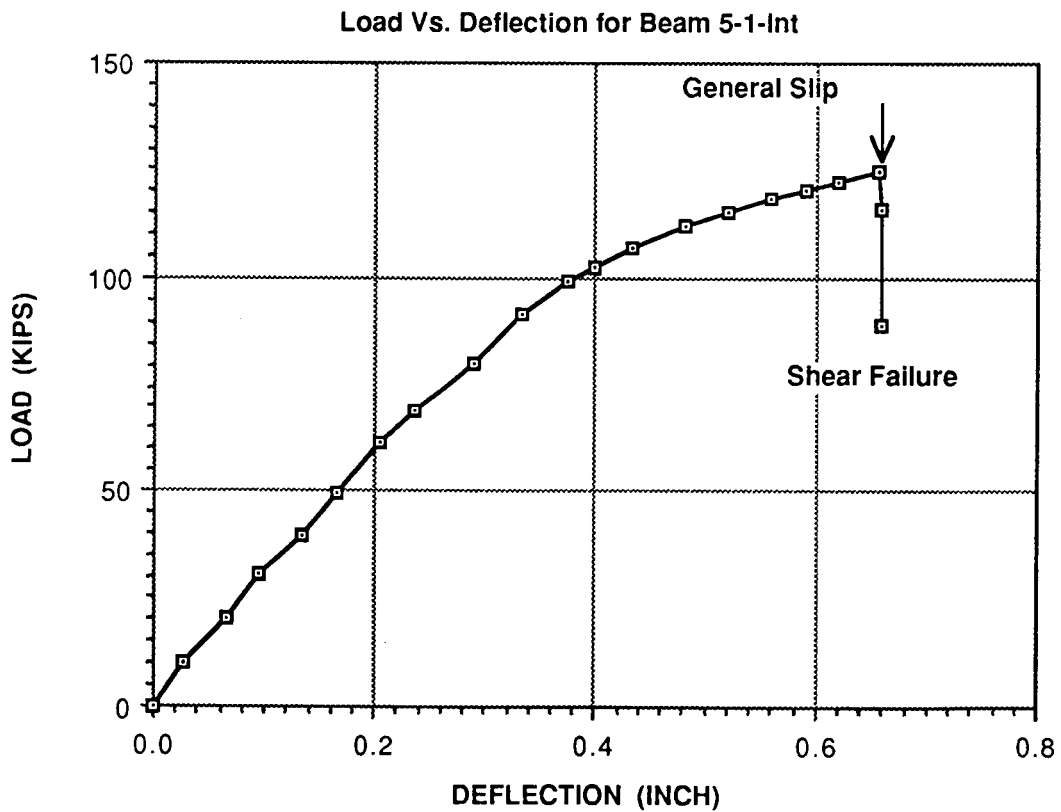
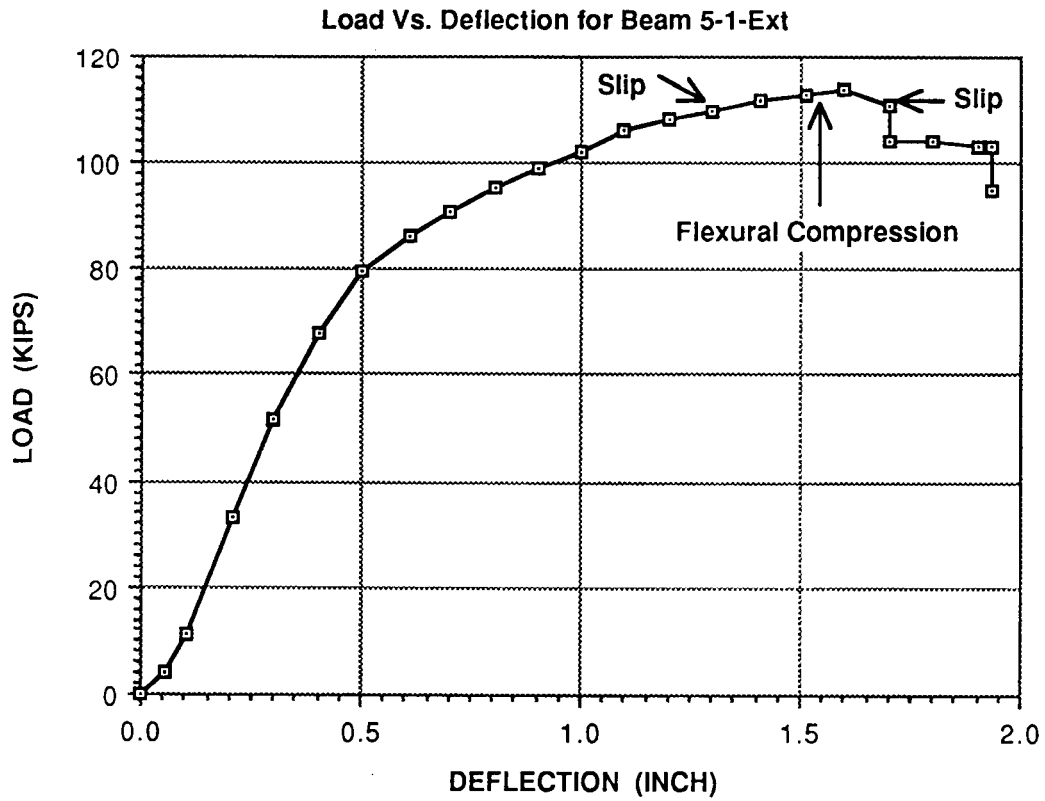


APPENDIX C

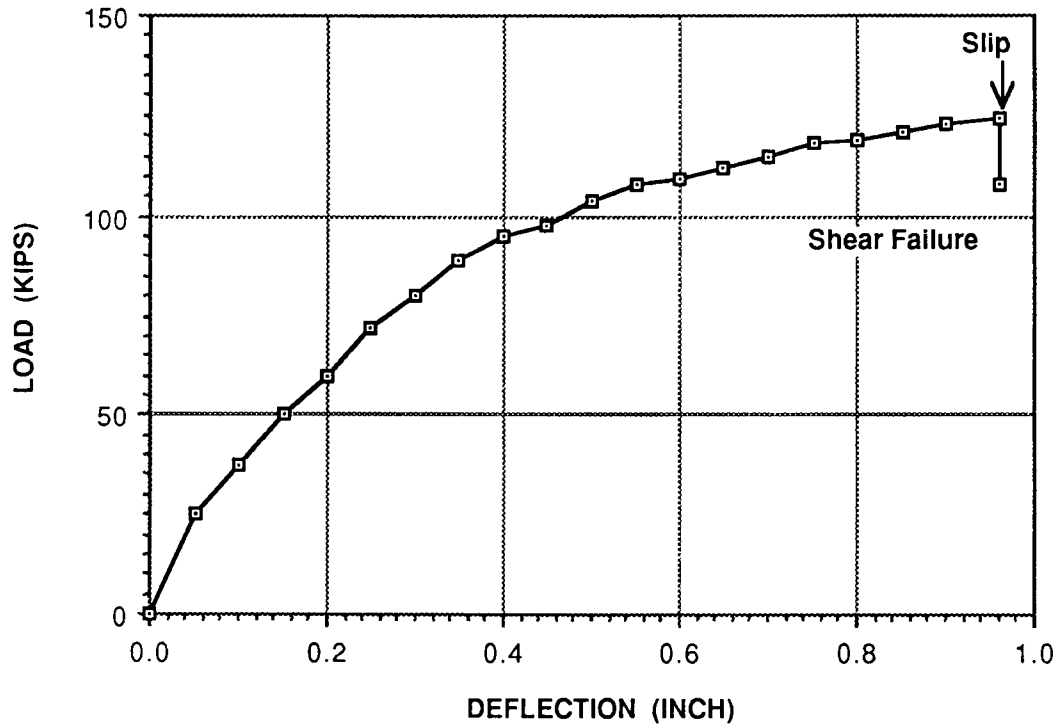
LOAD VERSUS DEFLECTION PLOTS

FOR

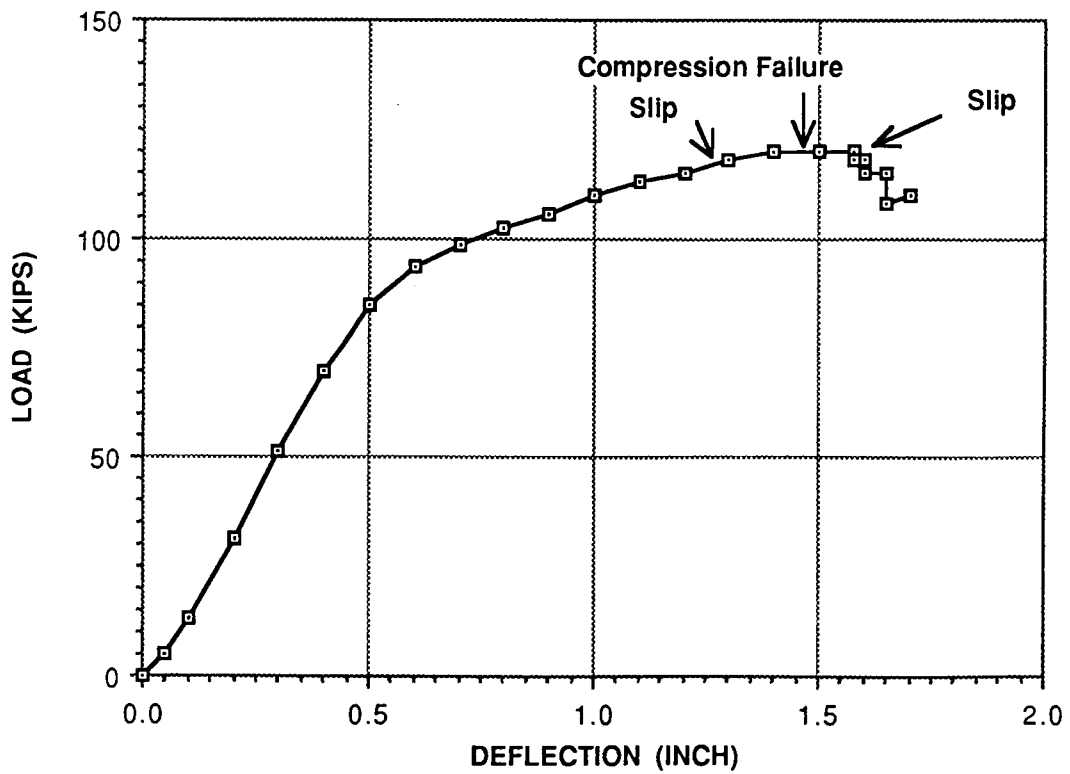
DEVELOPMENT LENGTH TESTS



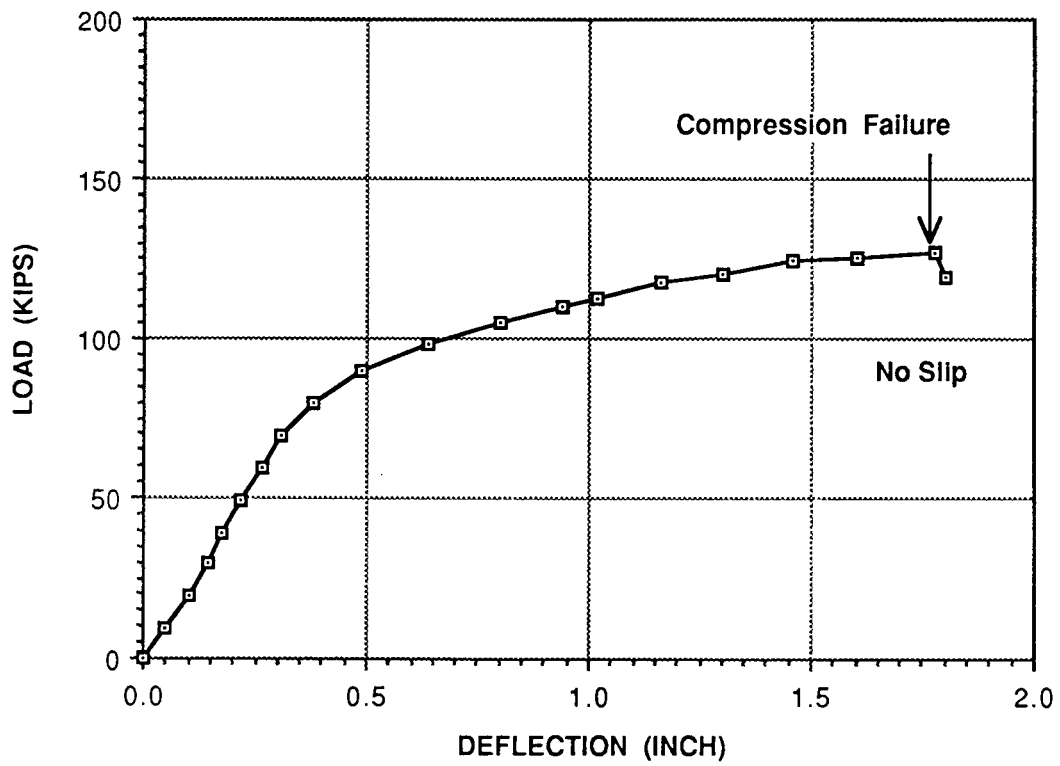
Load Vs. Deflection for Beam 5-2-EXT



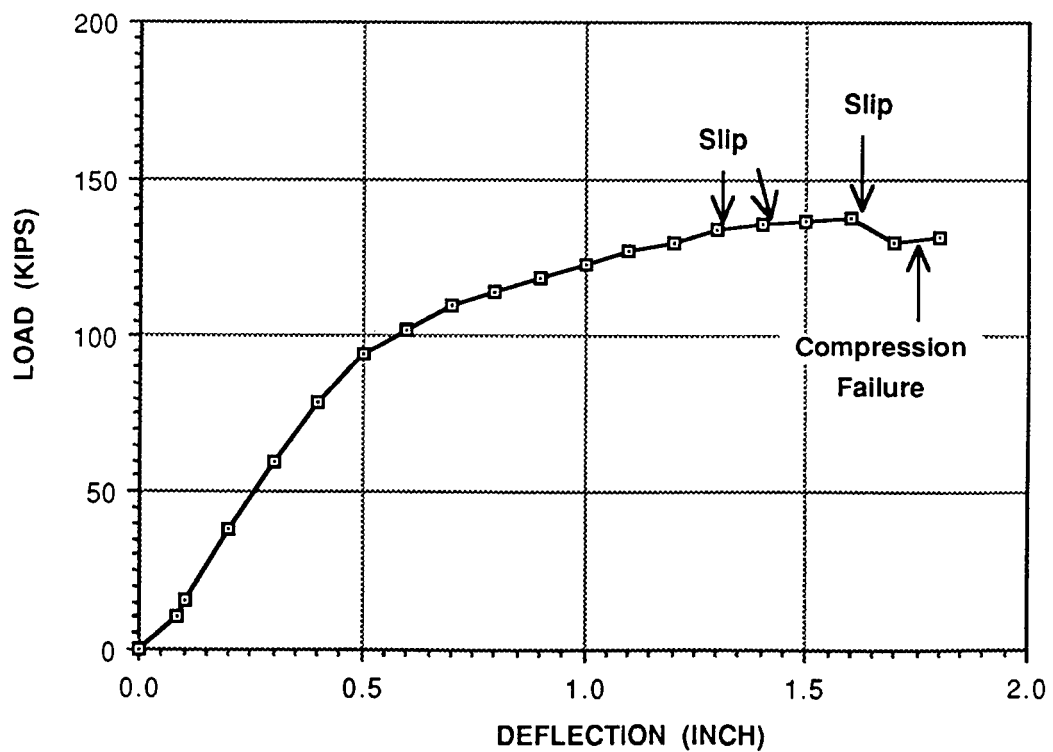
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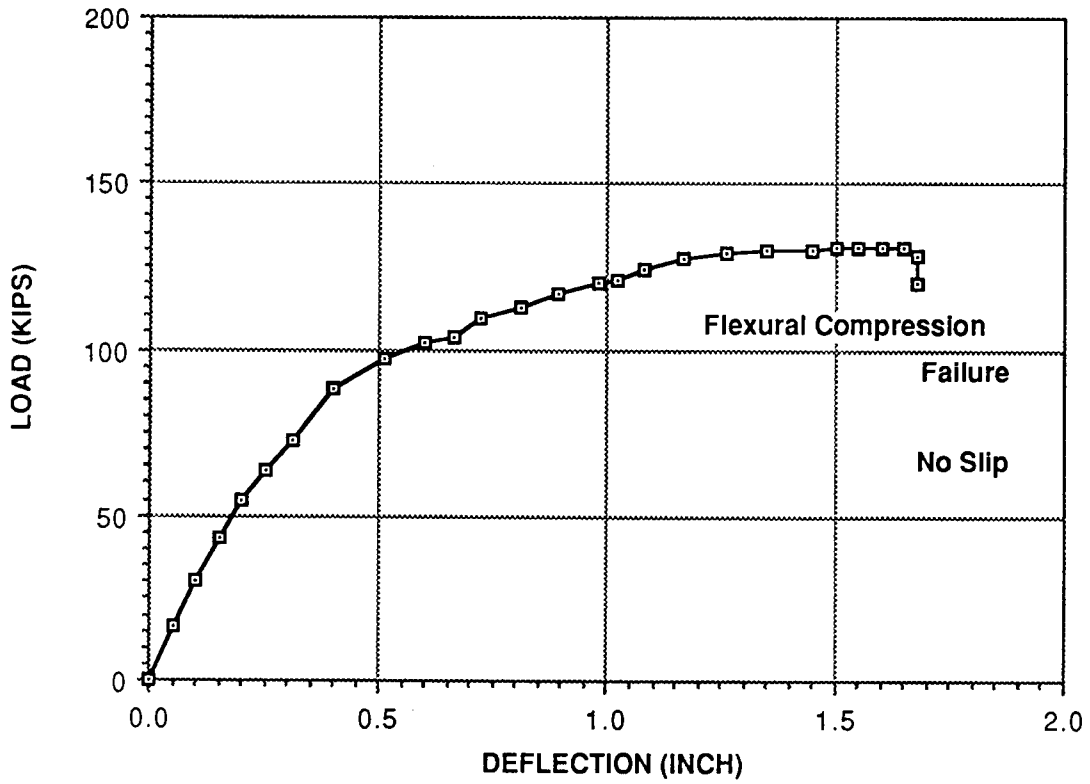
Load Vs. Deflection for Beam 5-3-EXT



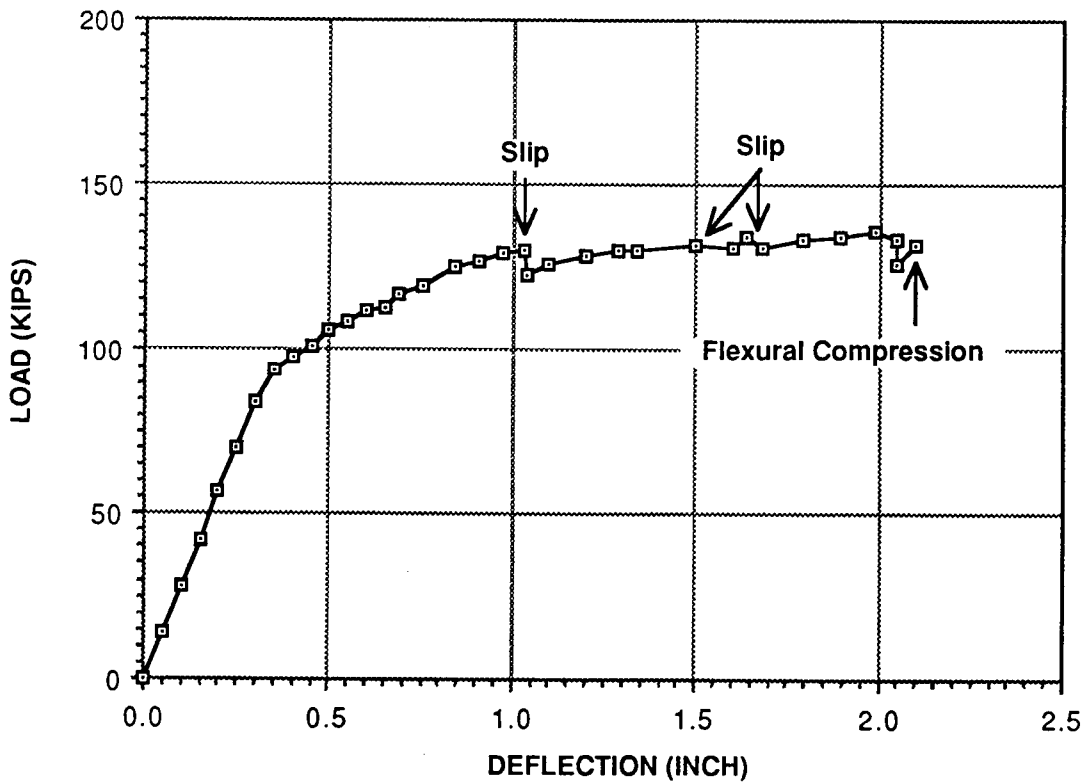
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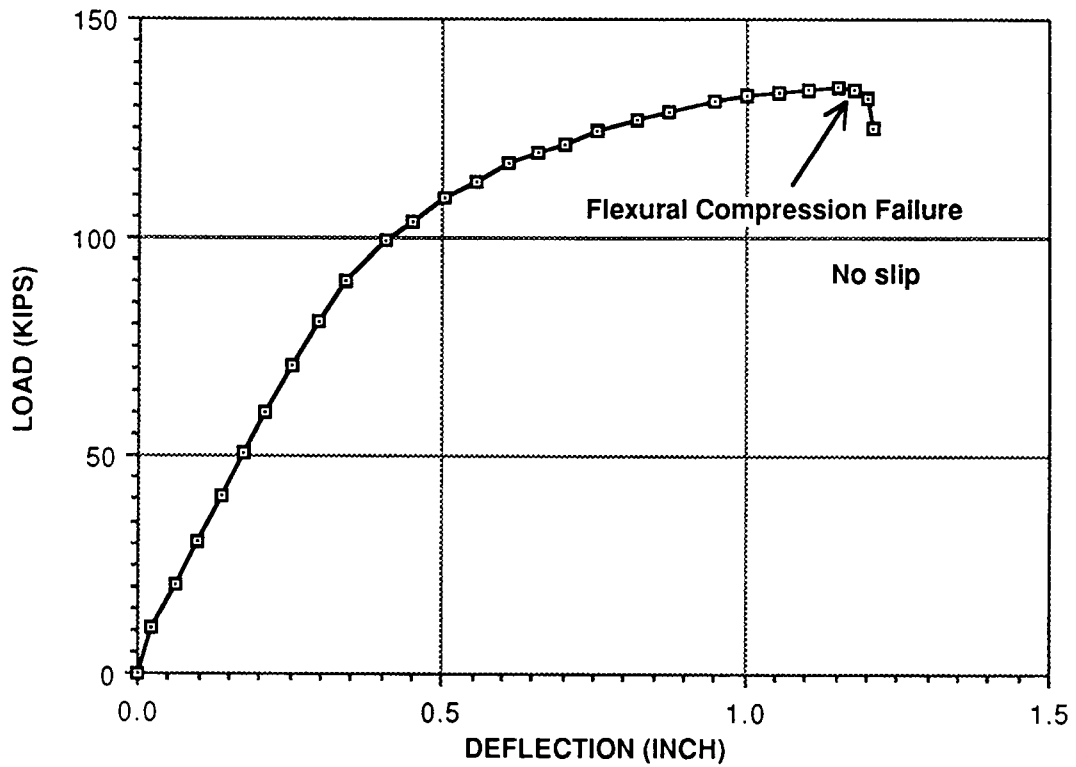
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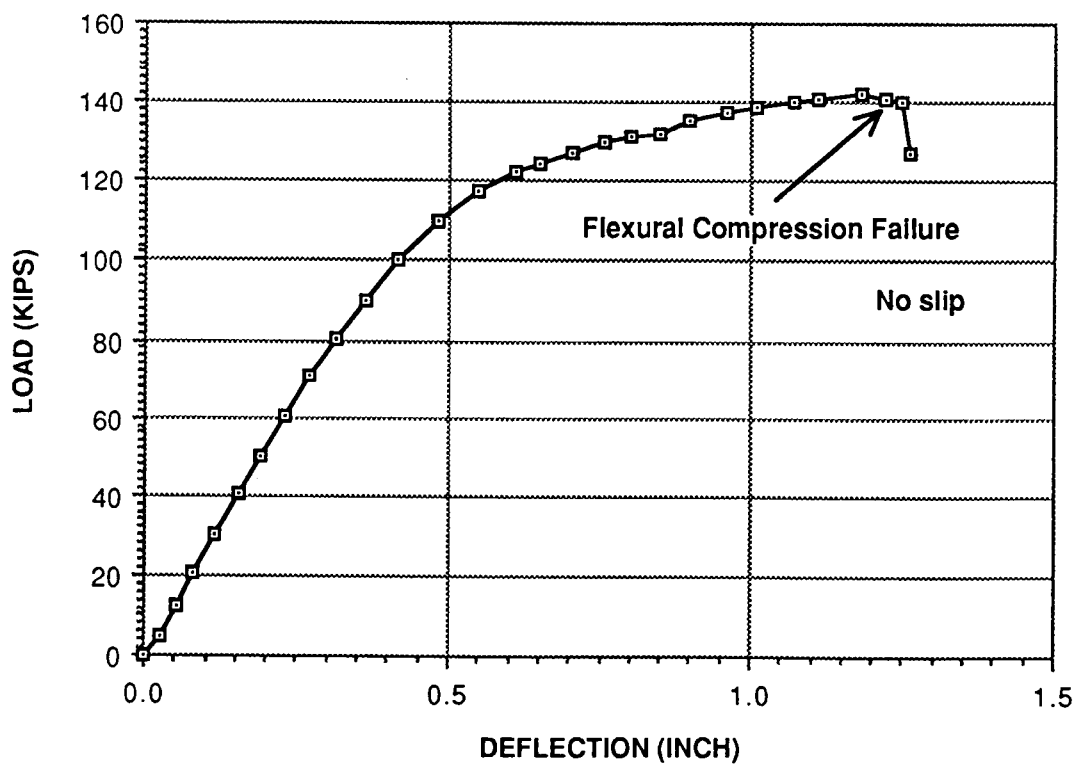
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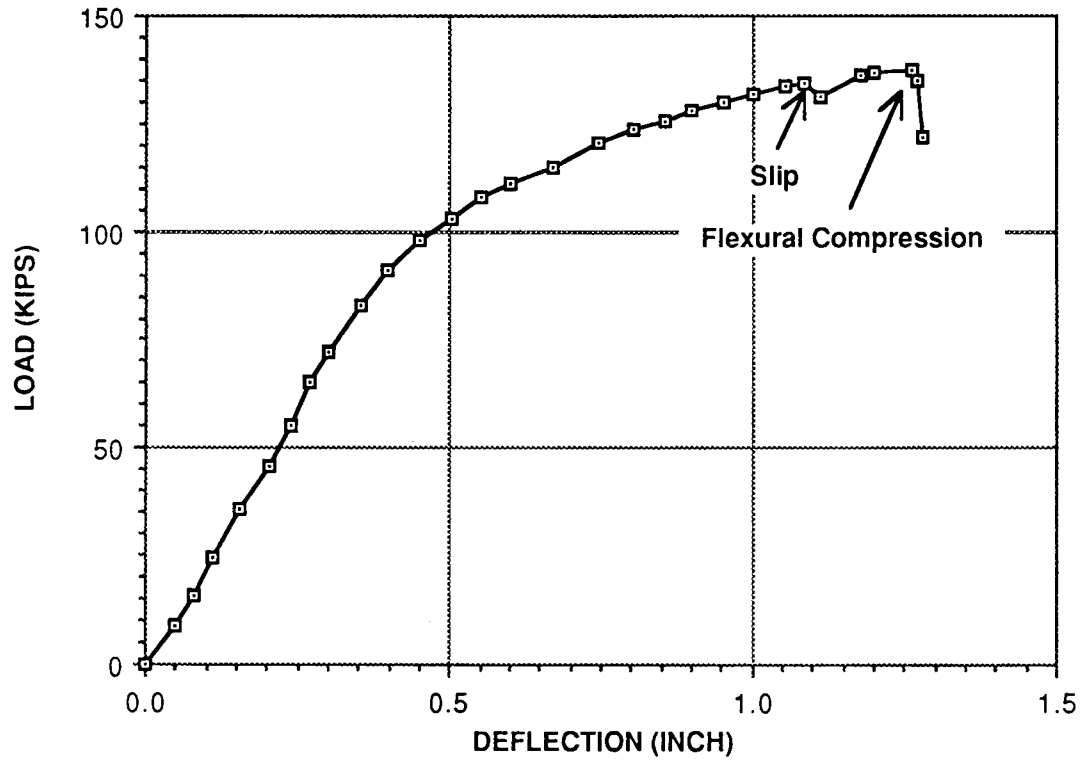
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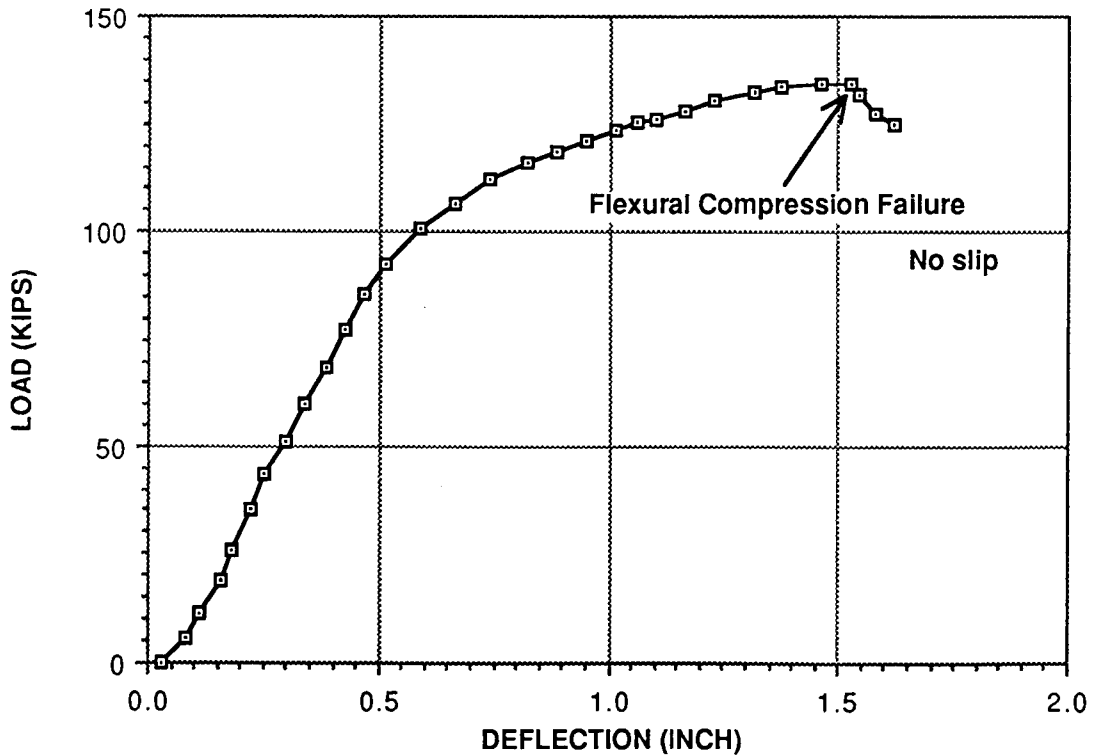
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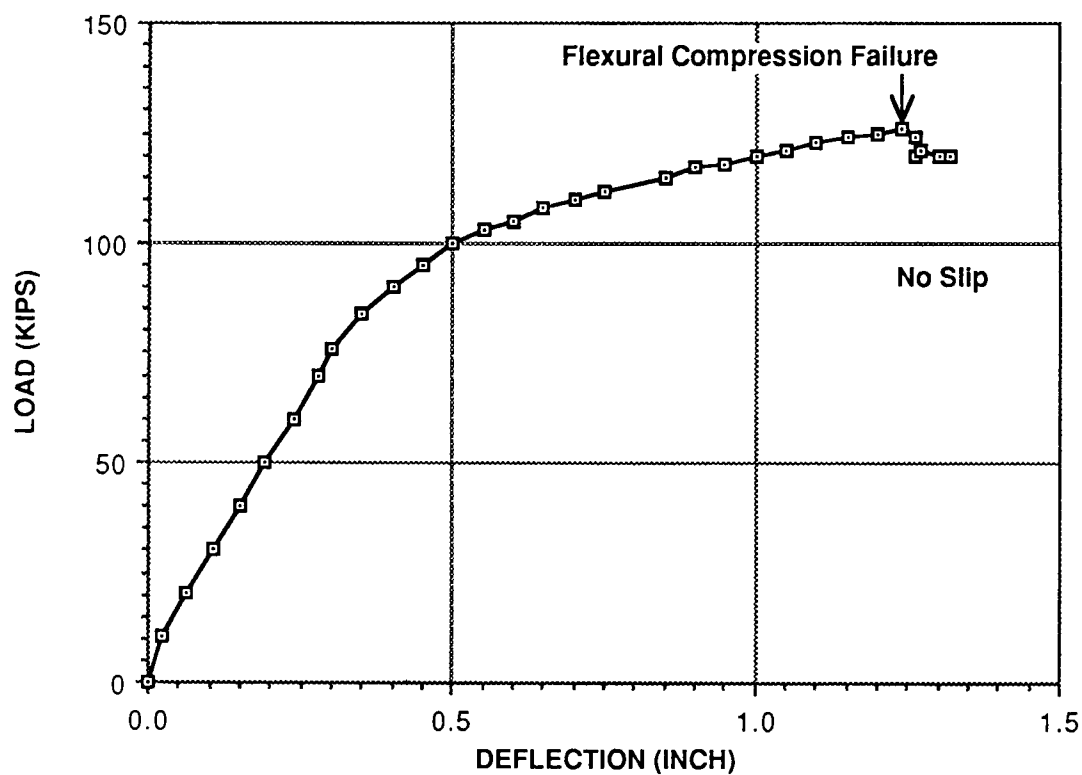
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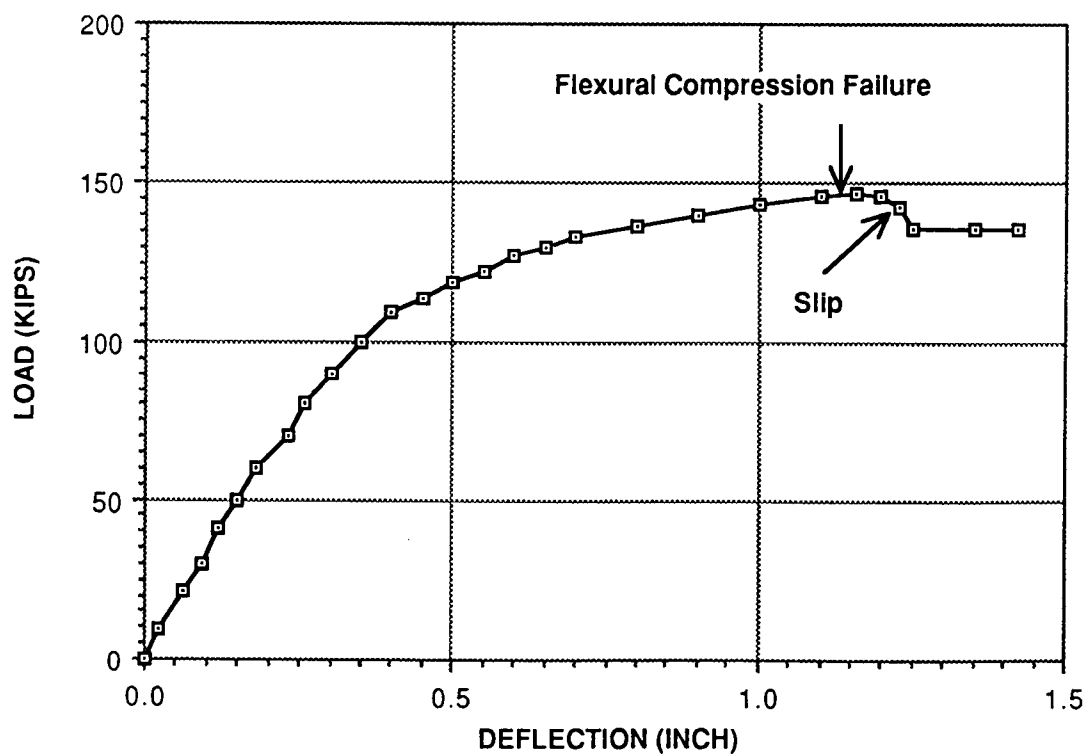
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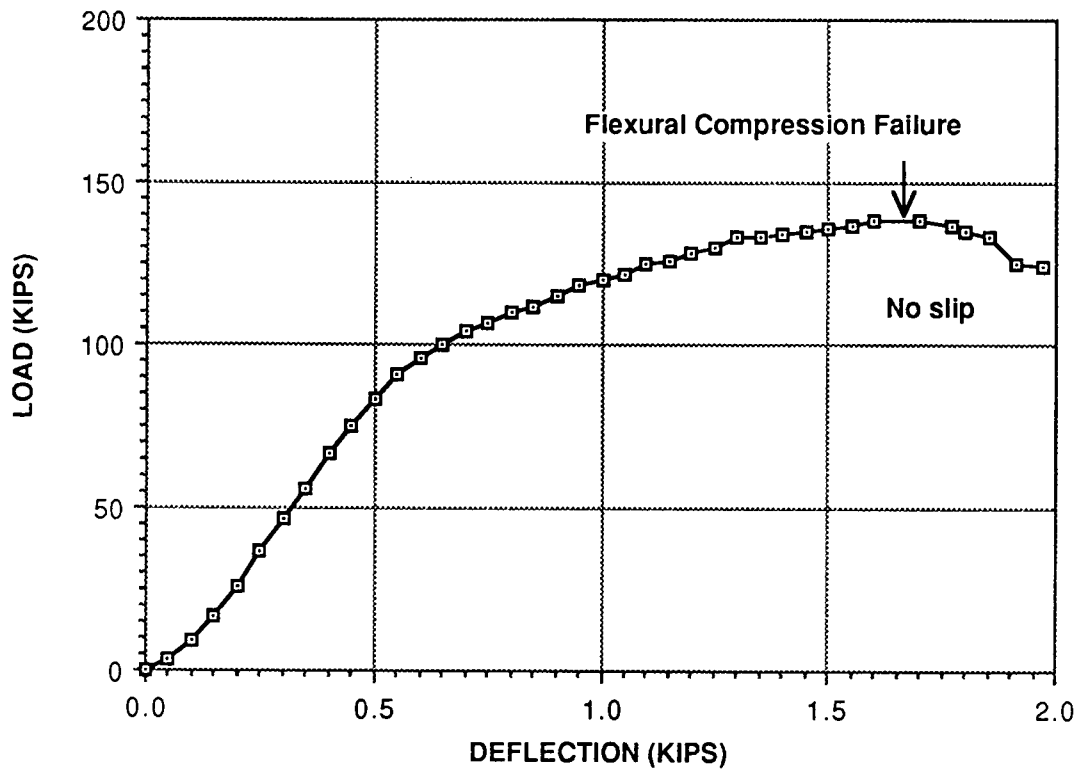
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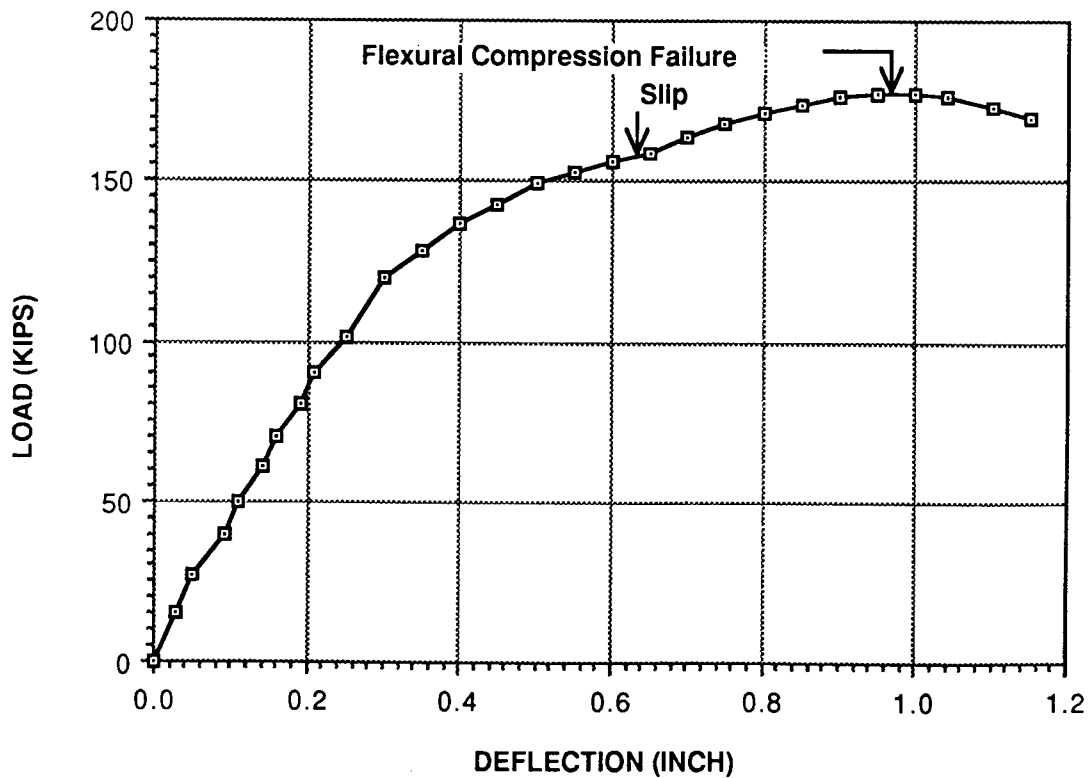
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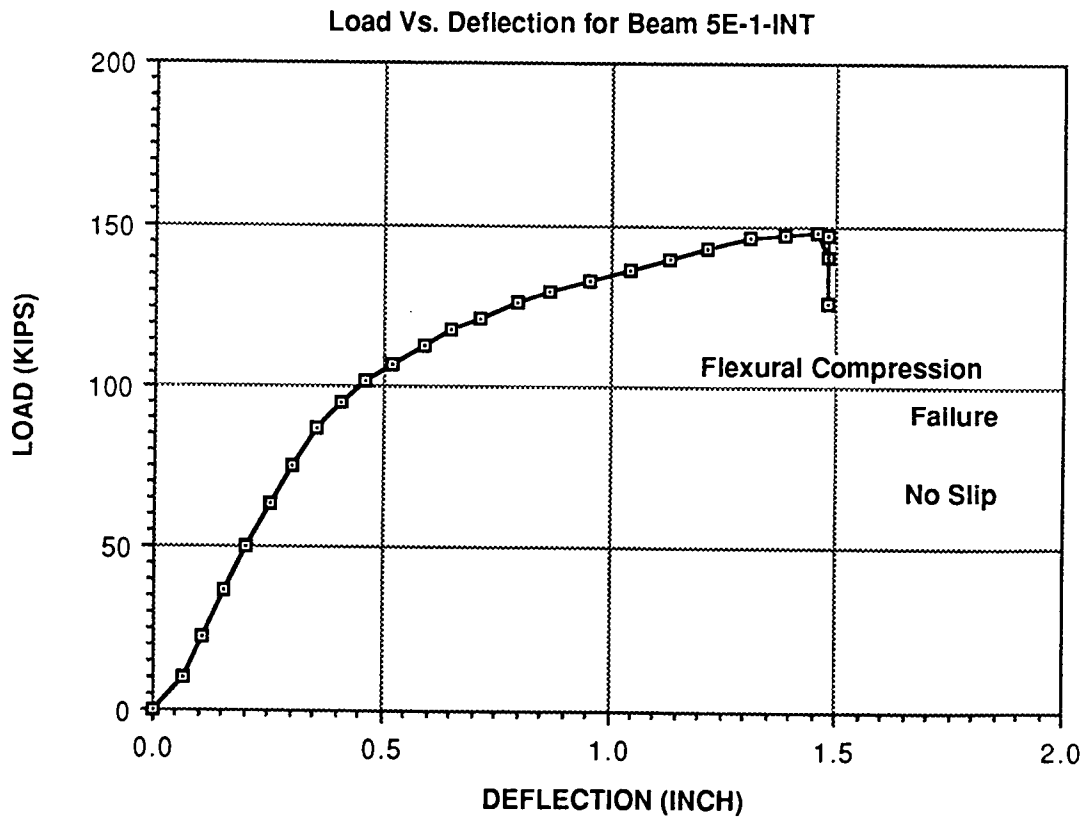
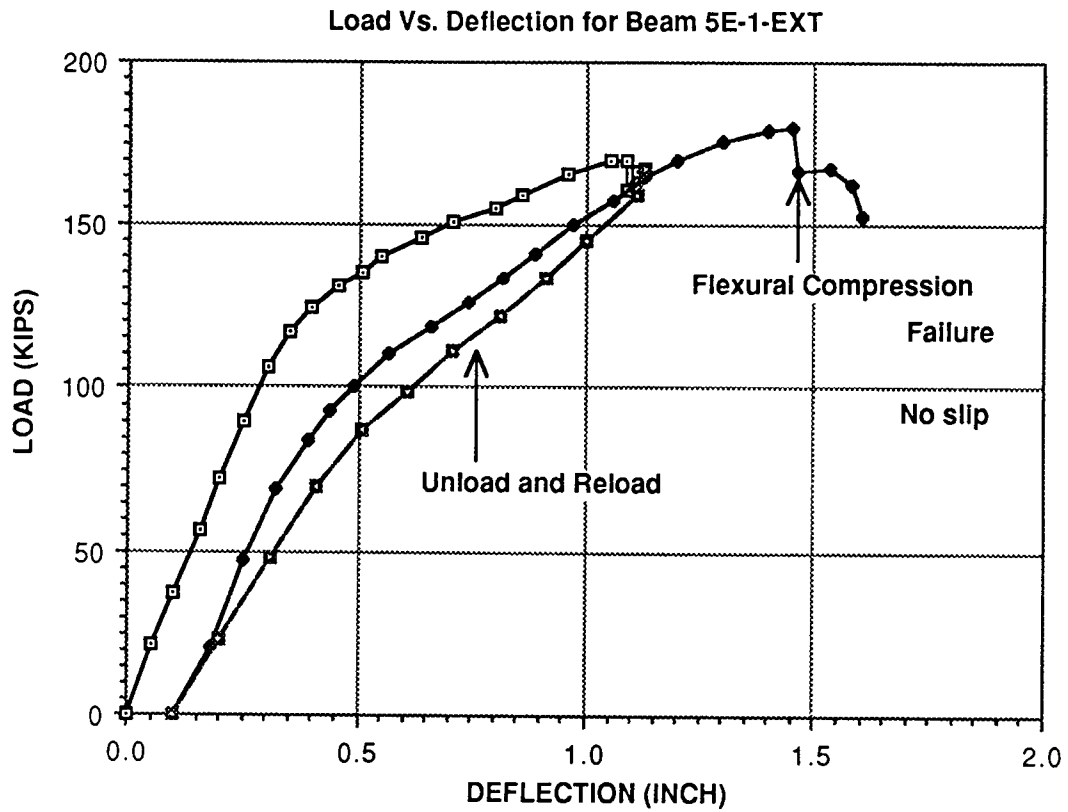


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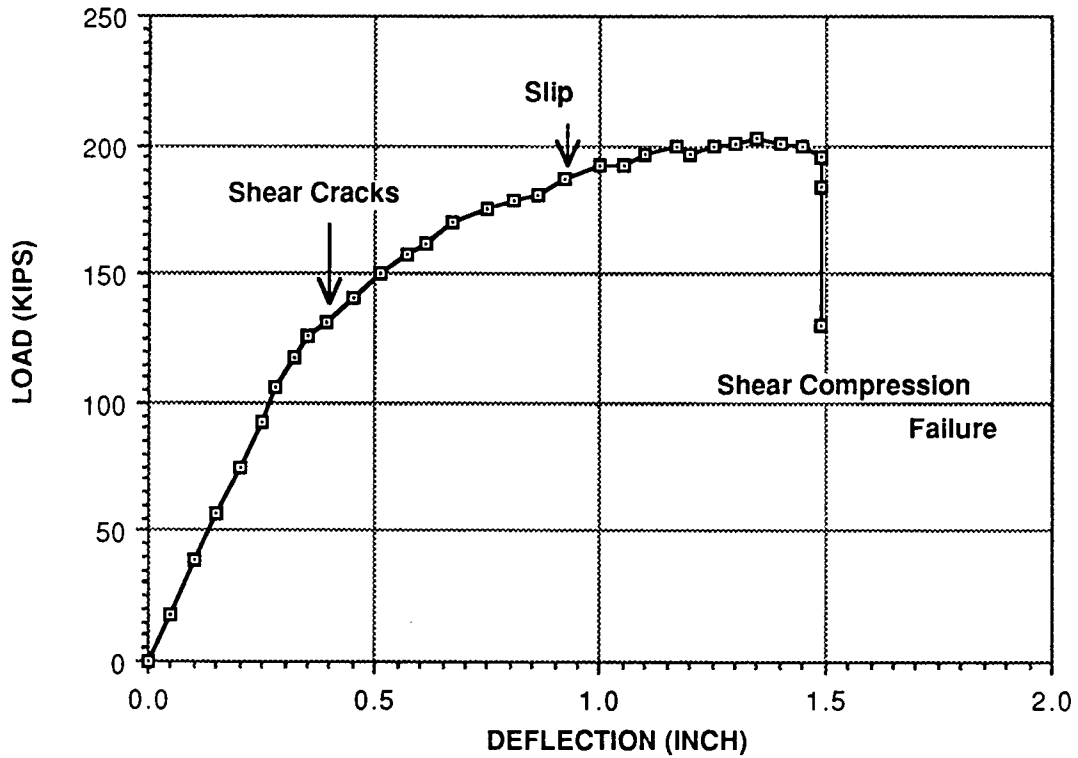


Load Vs. Deflection for Beam 5-UWR-WEST

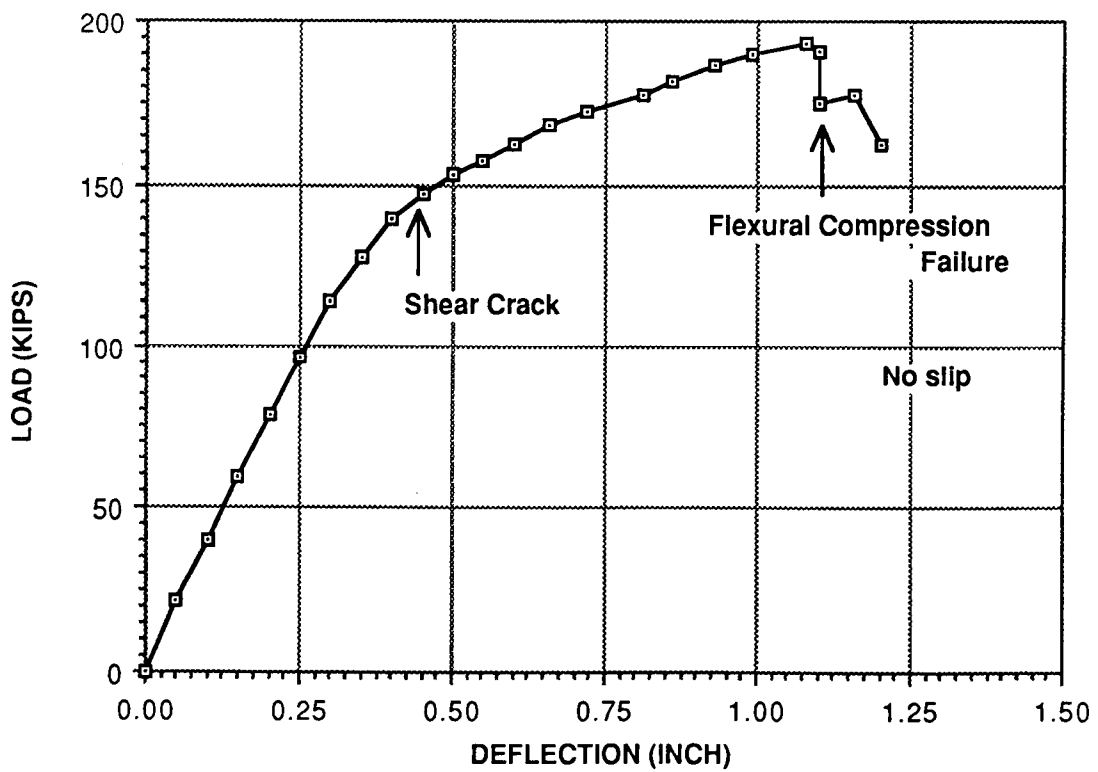


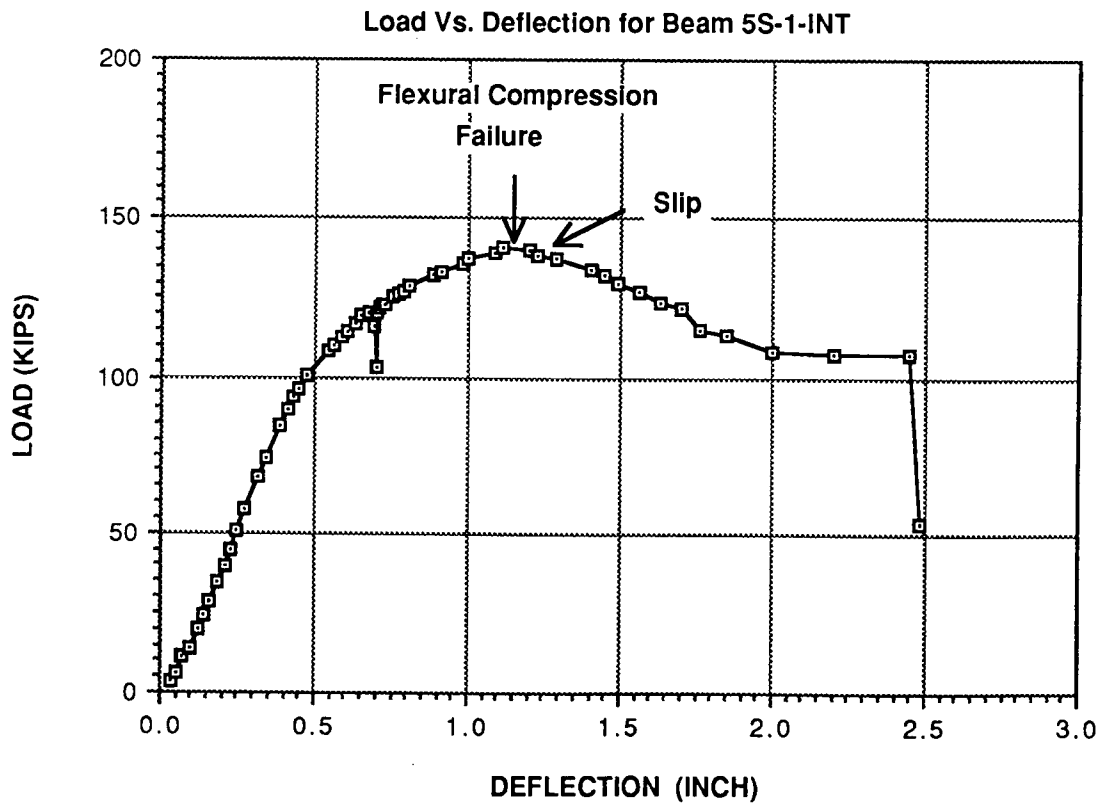
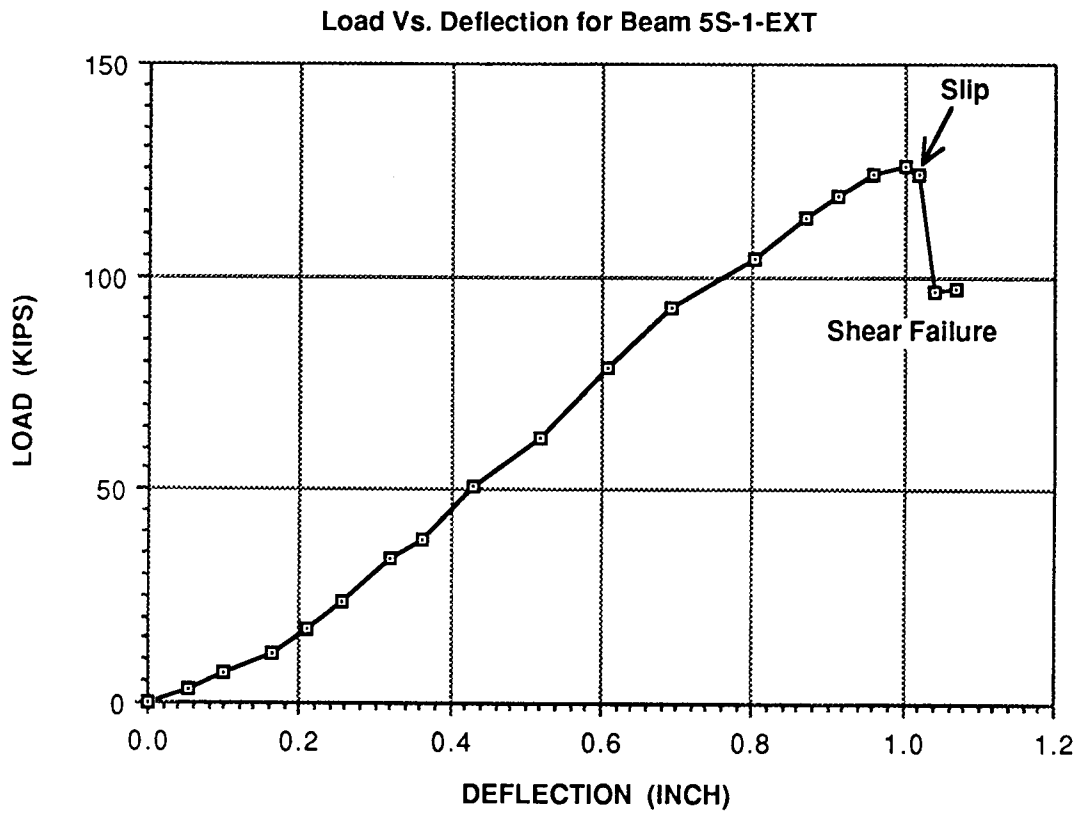


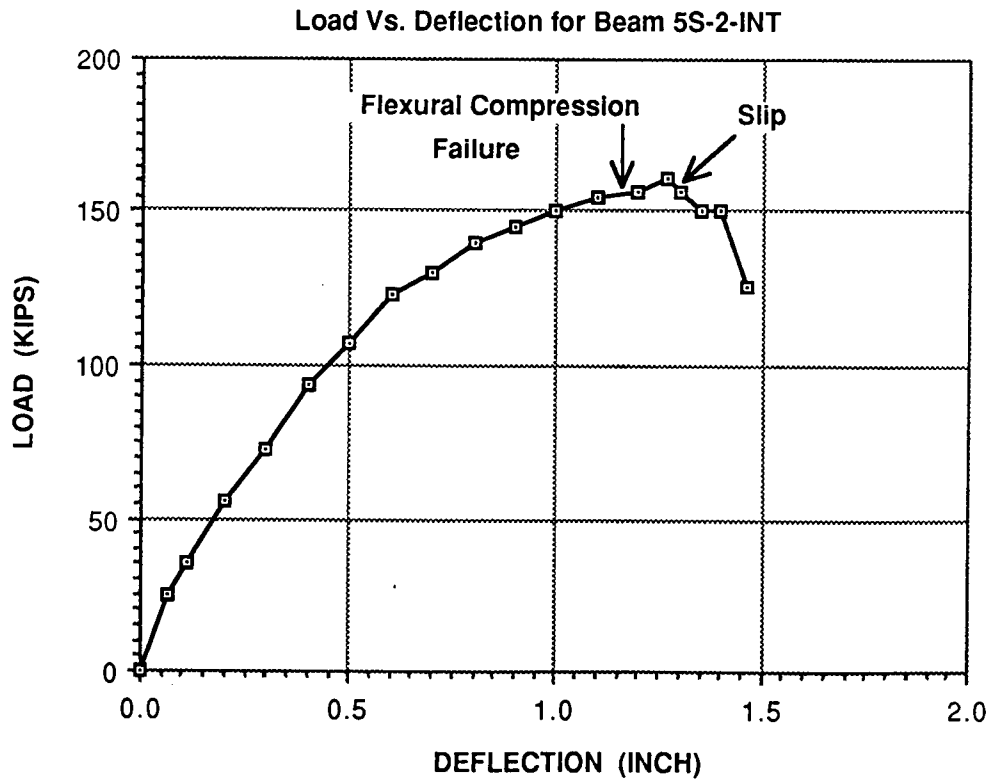
Load Vs. Deflection for Beam 5E-2-EXT



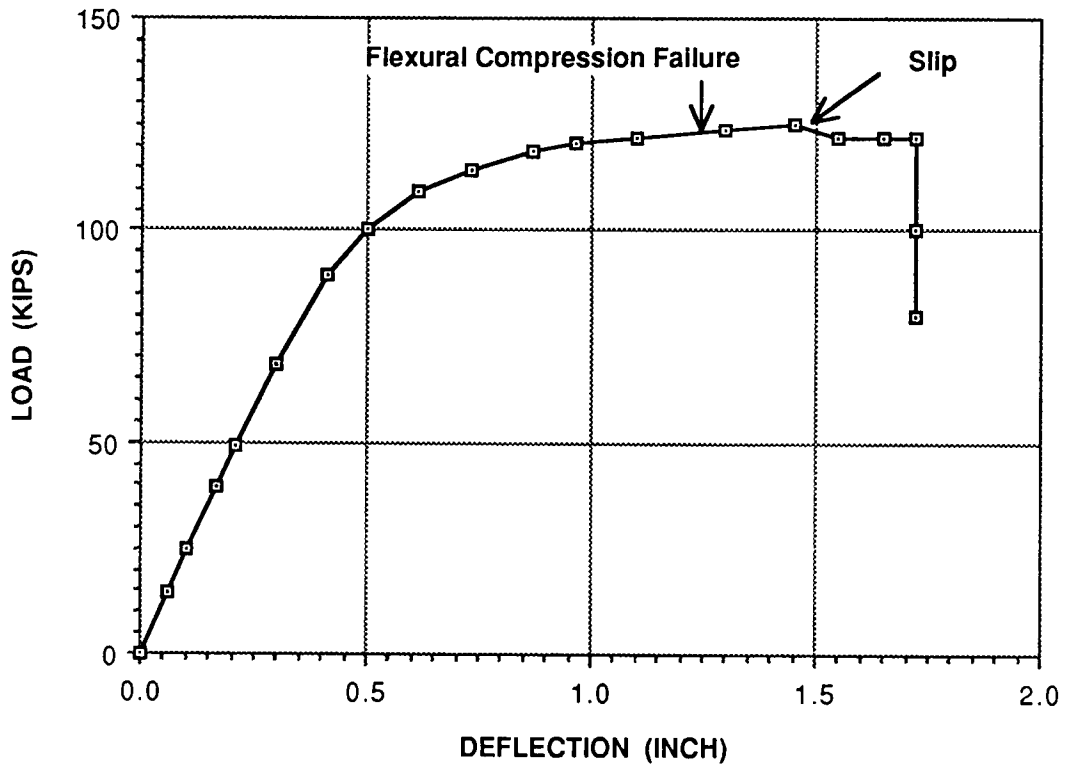
Load Vs. Deflection for Beam 5E-2-INT



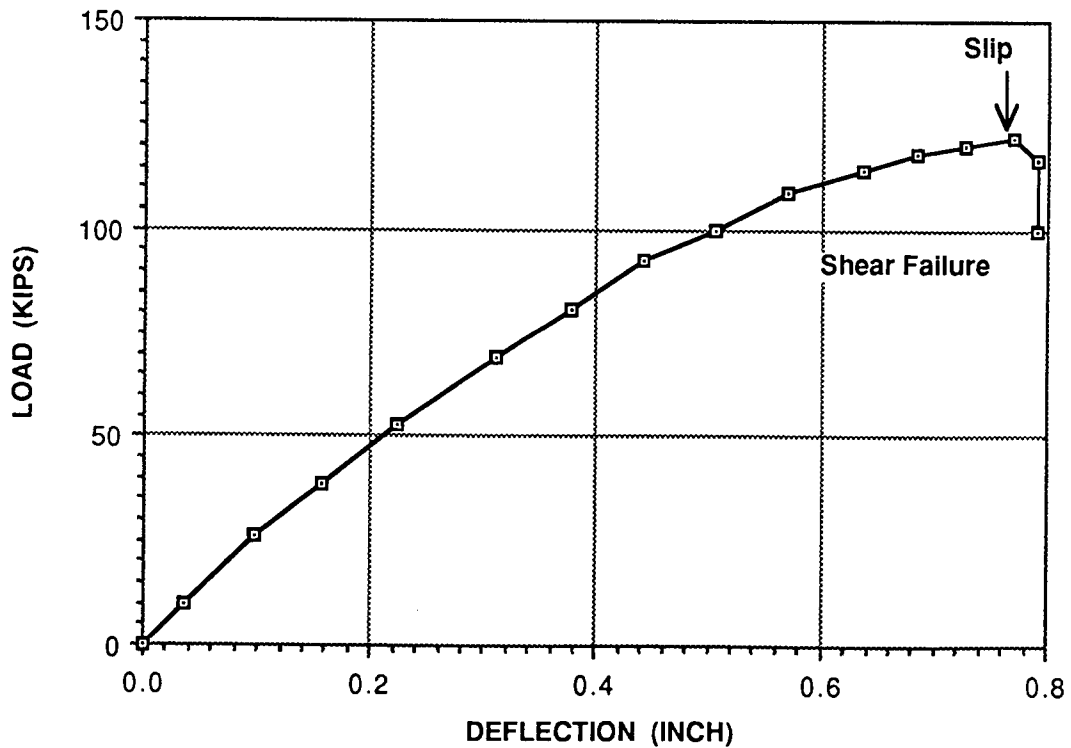




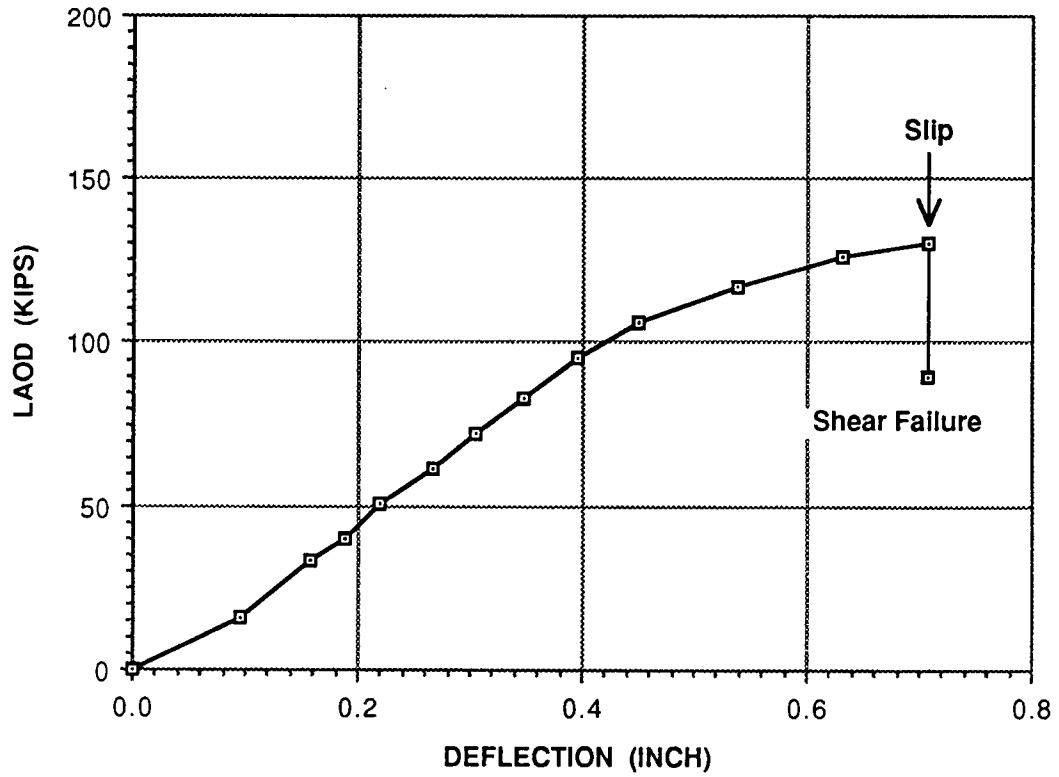
Load Vs. Deflection for Beam 5S-3-EXT



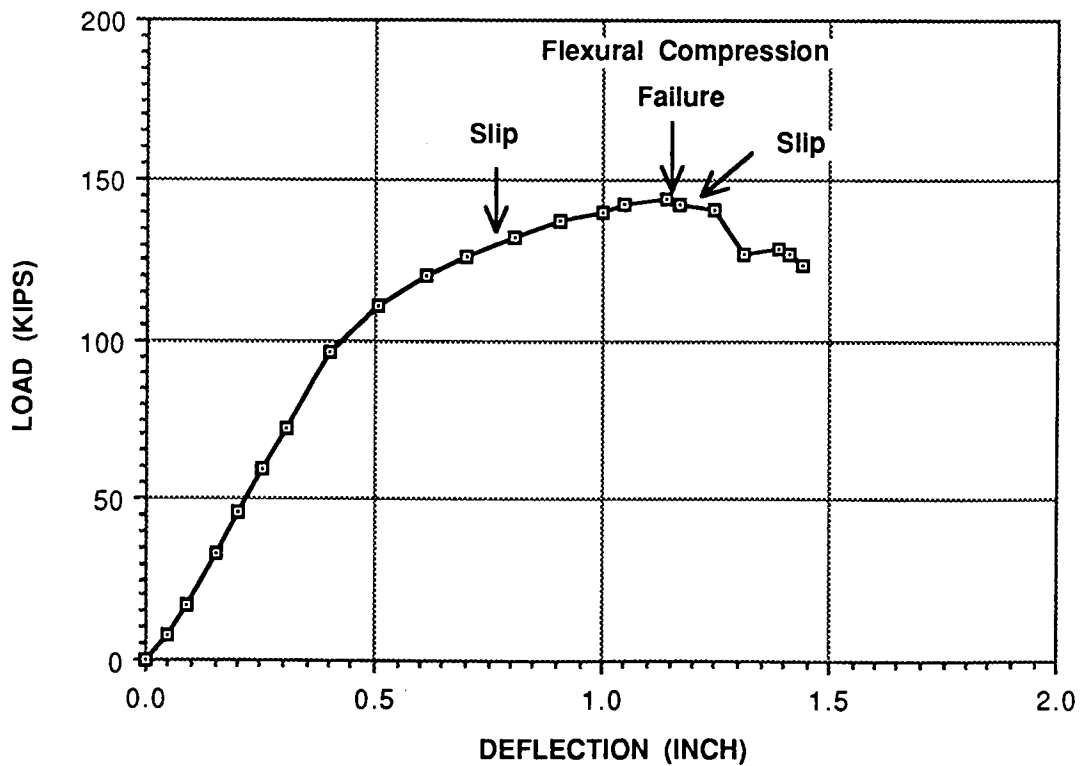
Load Vs. Deflection for Beam 5S-3-INT



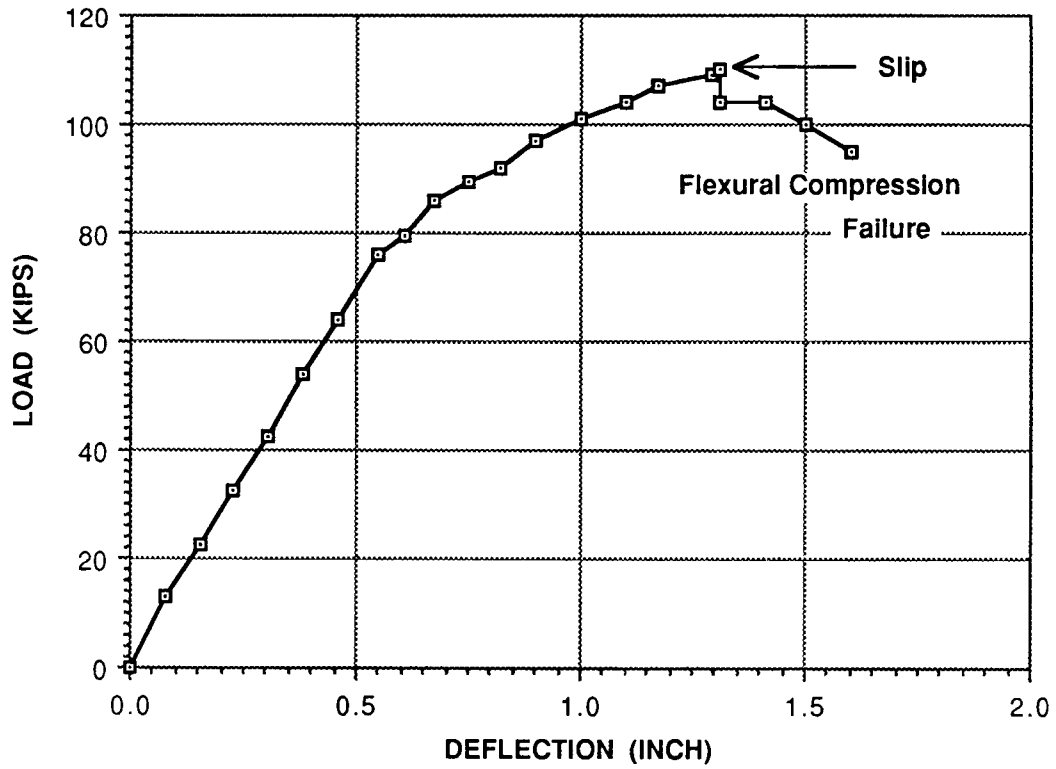
Load Vs. Deflection for Beam 5S-4-EXT



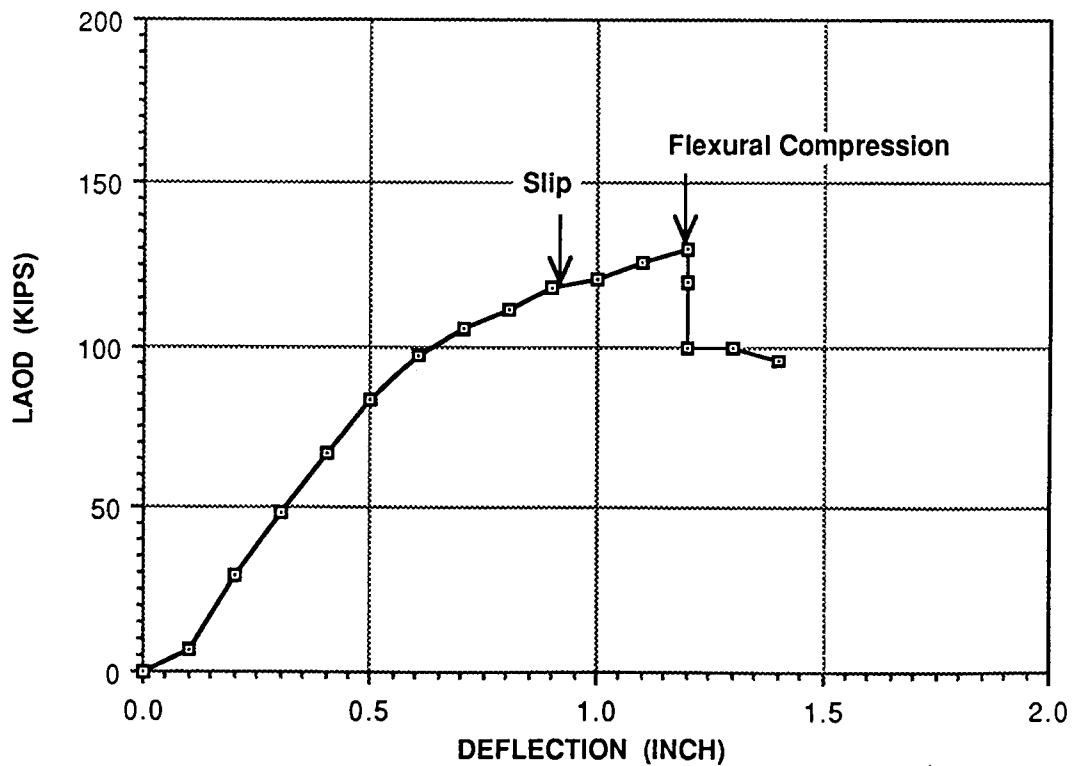
Load Vs. Deflection for Beam 5S-4-INT



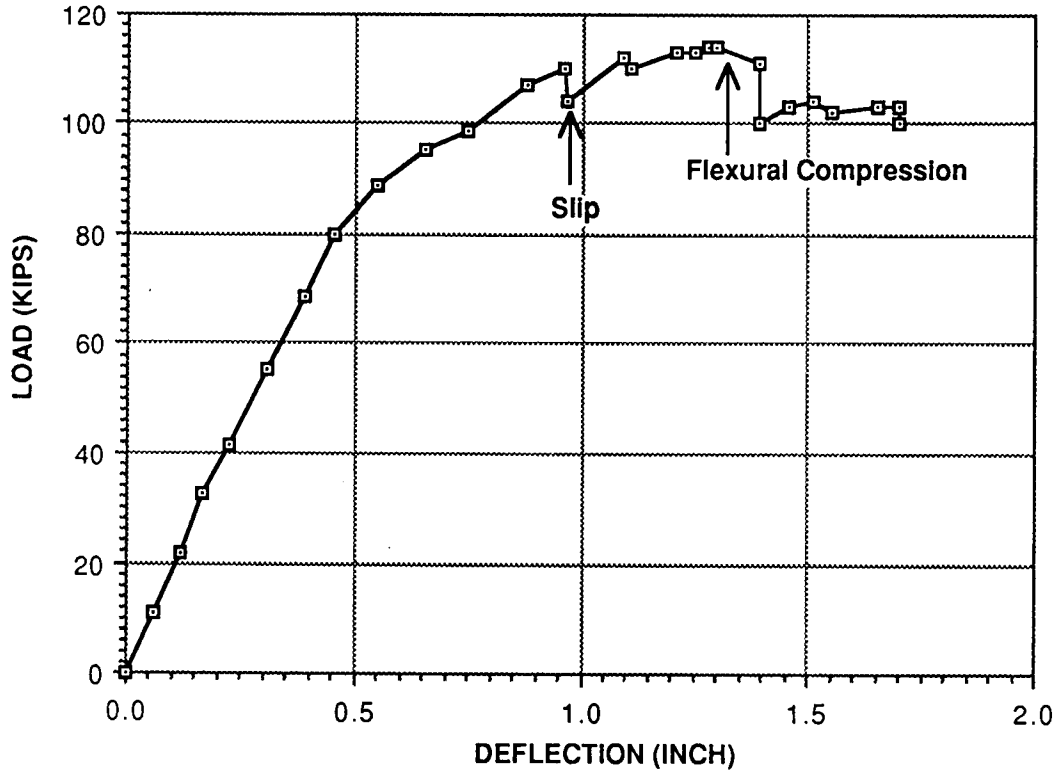
Load Vs. Deflection for Beam 916-1-Ext



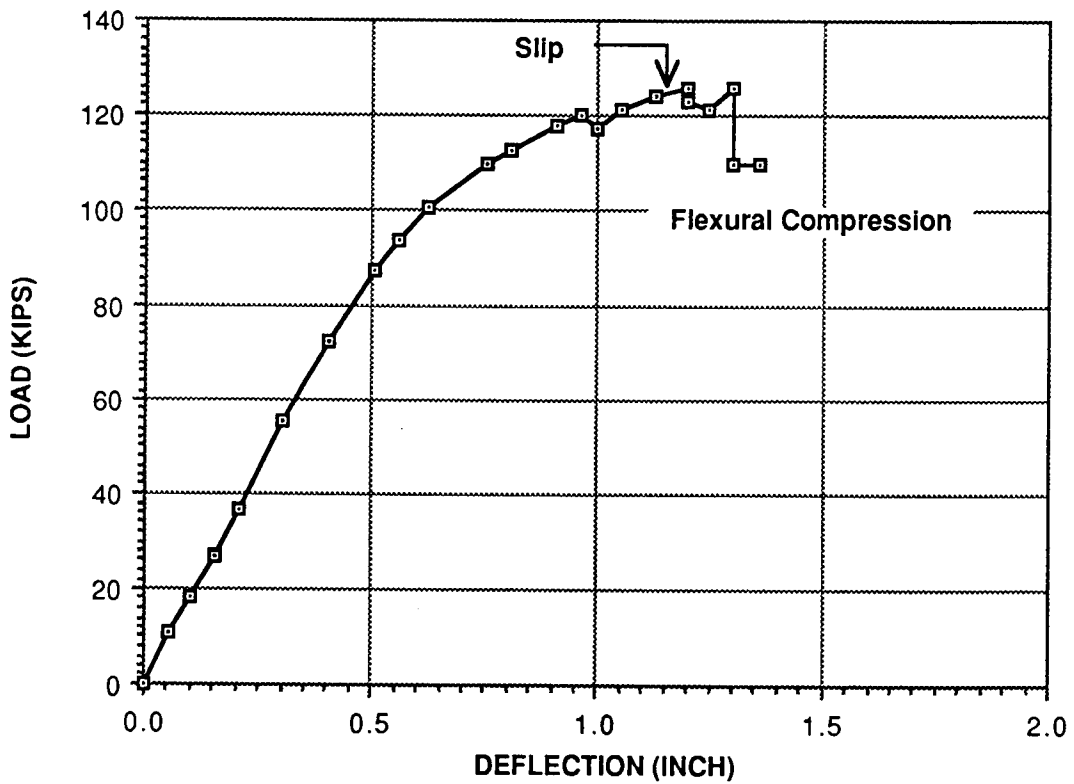
Load Vs. Deflection for Beam 916-1-Int



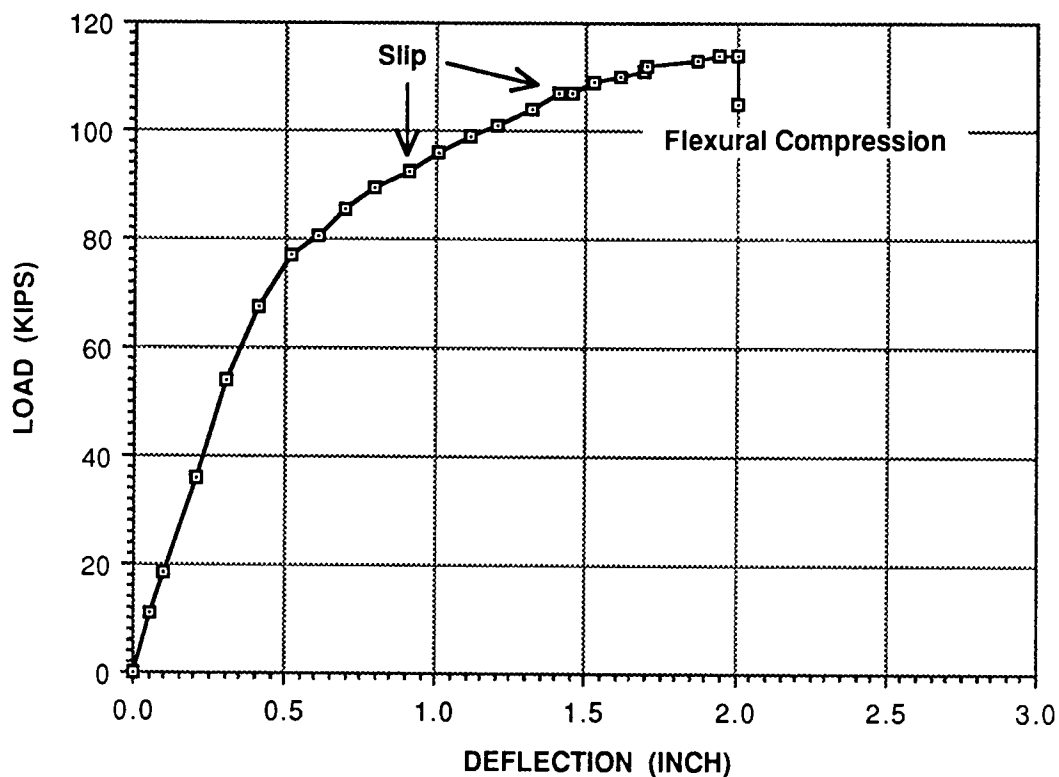
Load Vs. Deflection for Beam 916-2-EXT



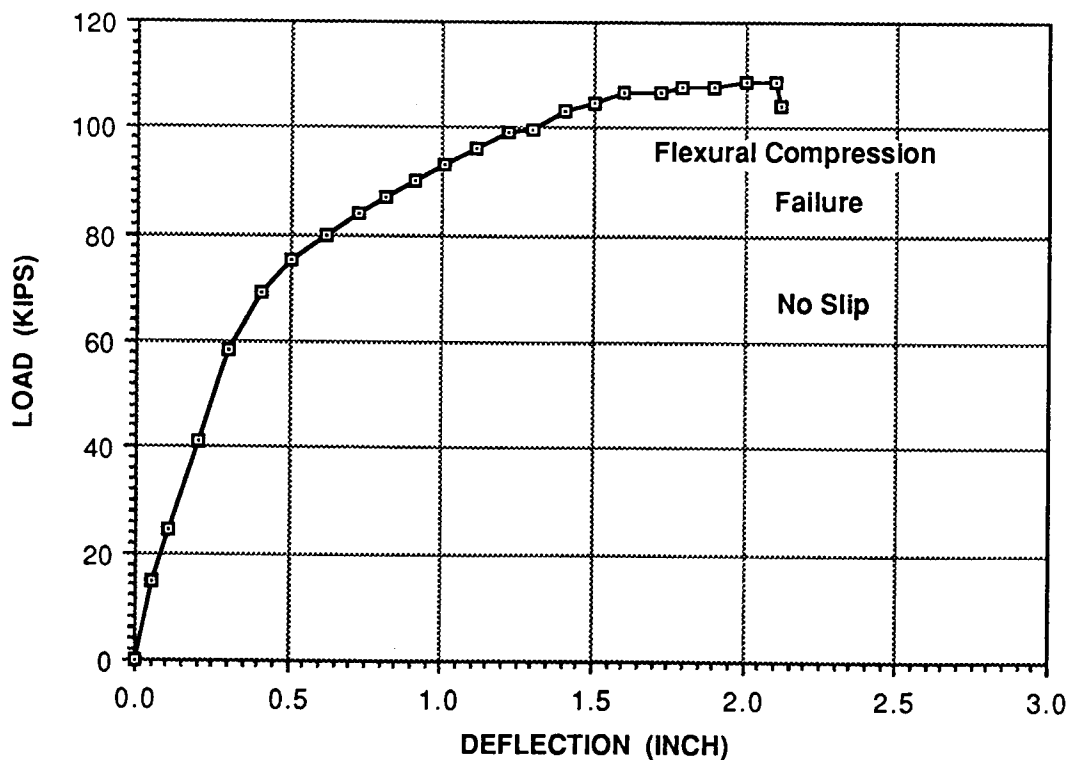
Load Vs. Deflection for Beam 916-2-Int



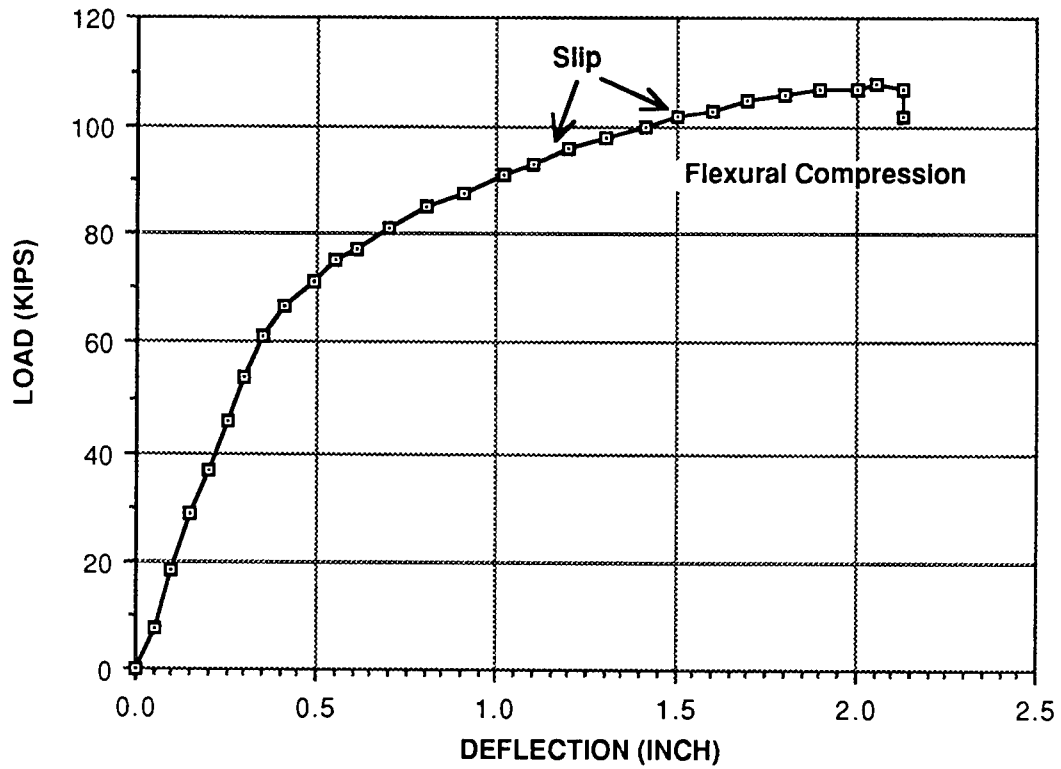
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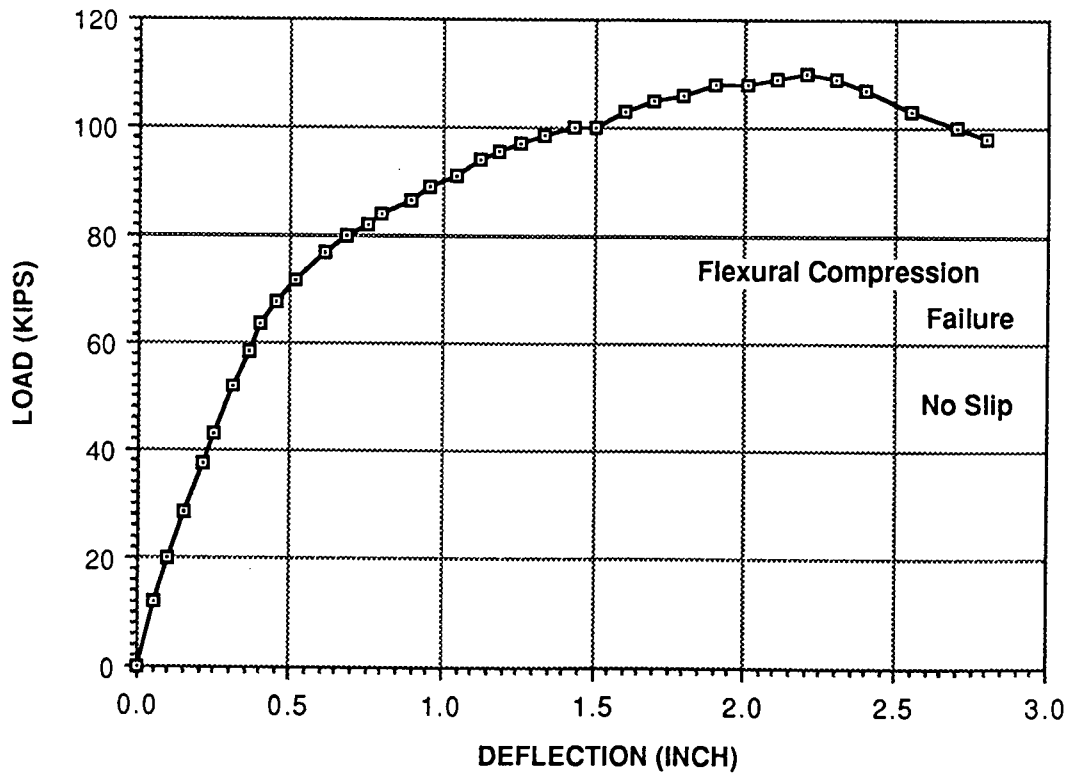
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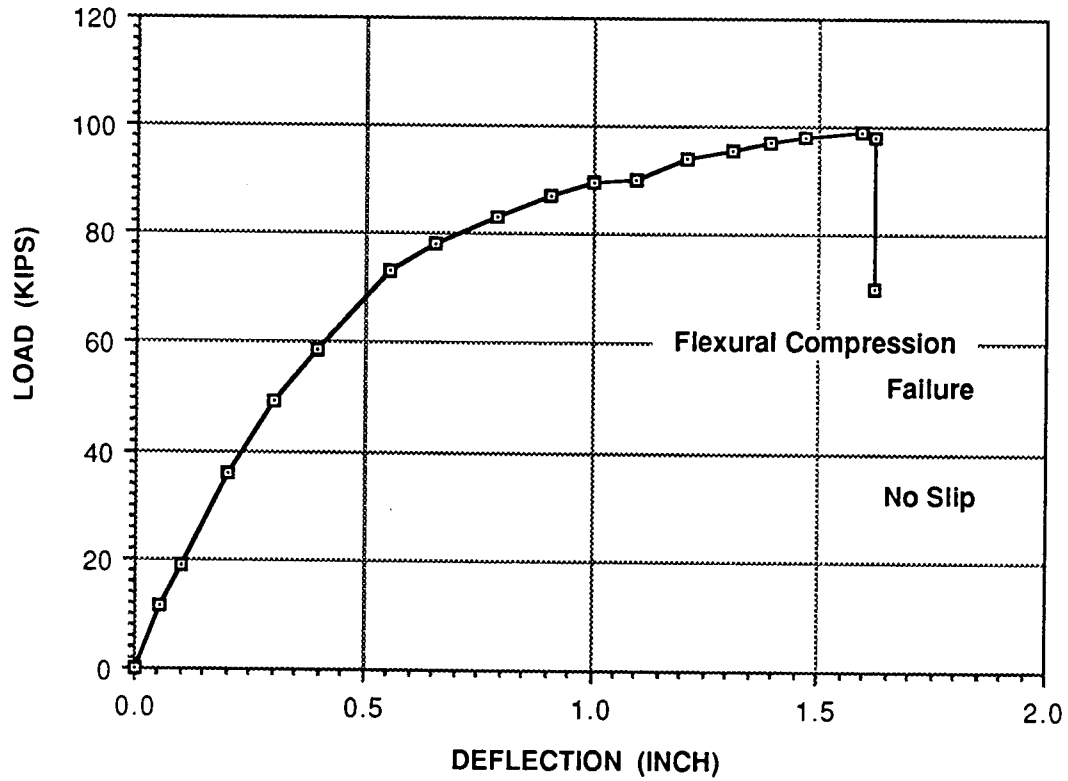
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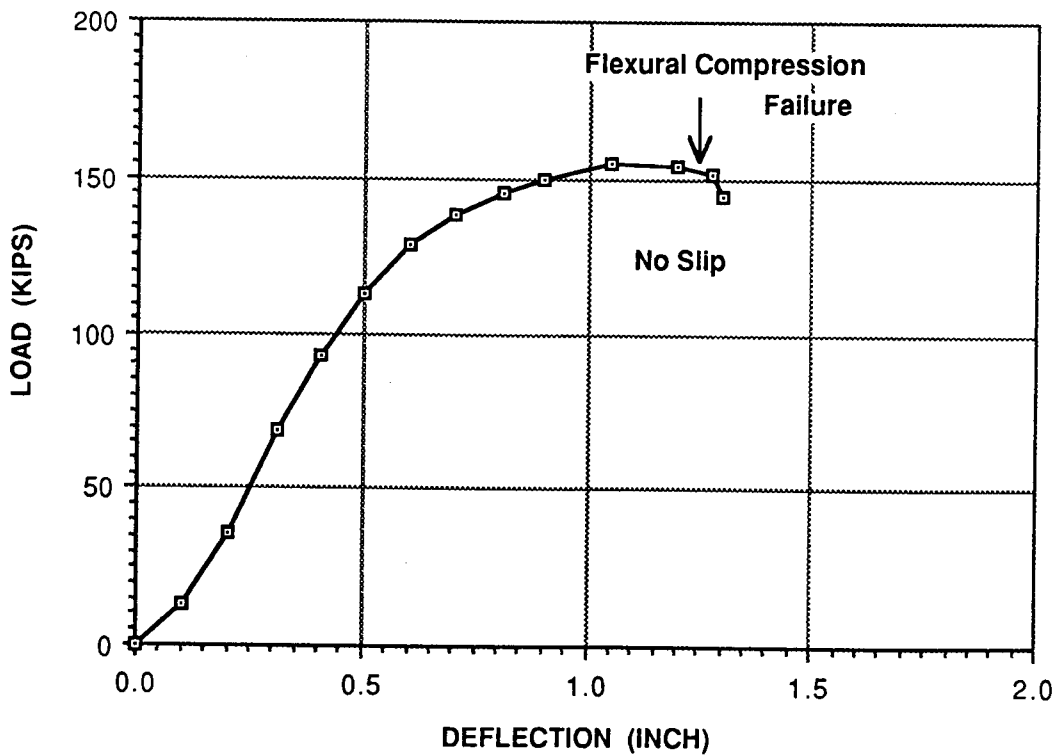
Load Vs. Deflection for Beam 916-4-INT



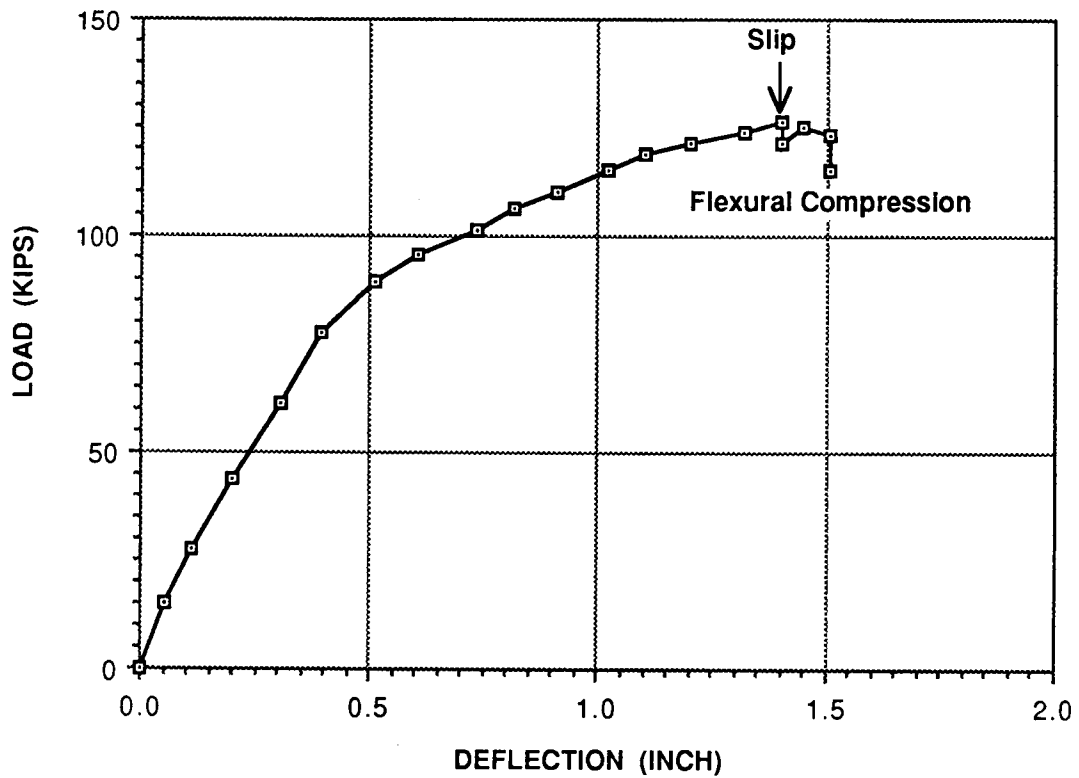
Load Vs. Deflection for Beam 6-1-Ext



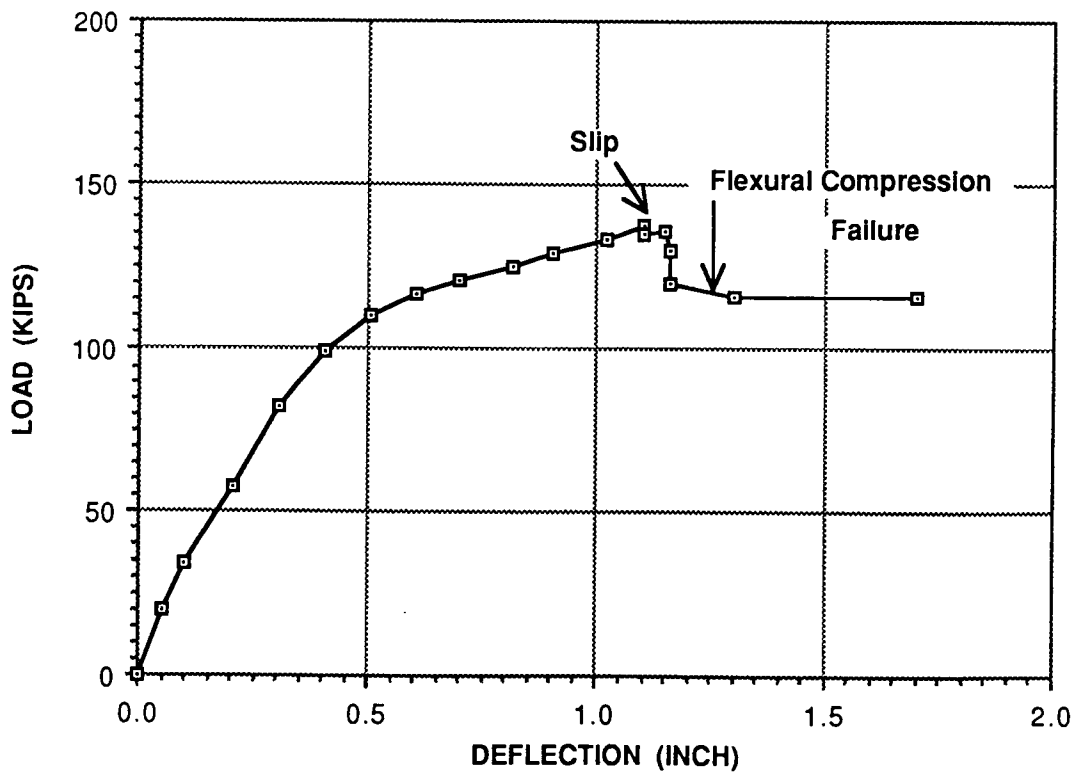
Load Vs. Deflection for Beam 6-1-Int



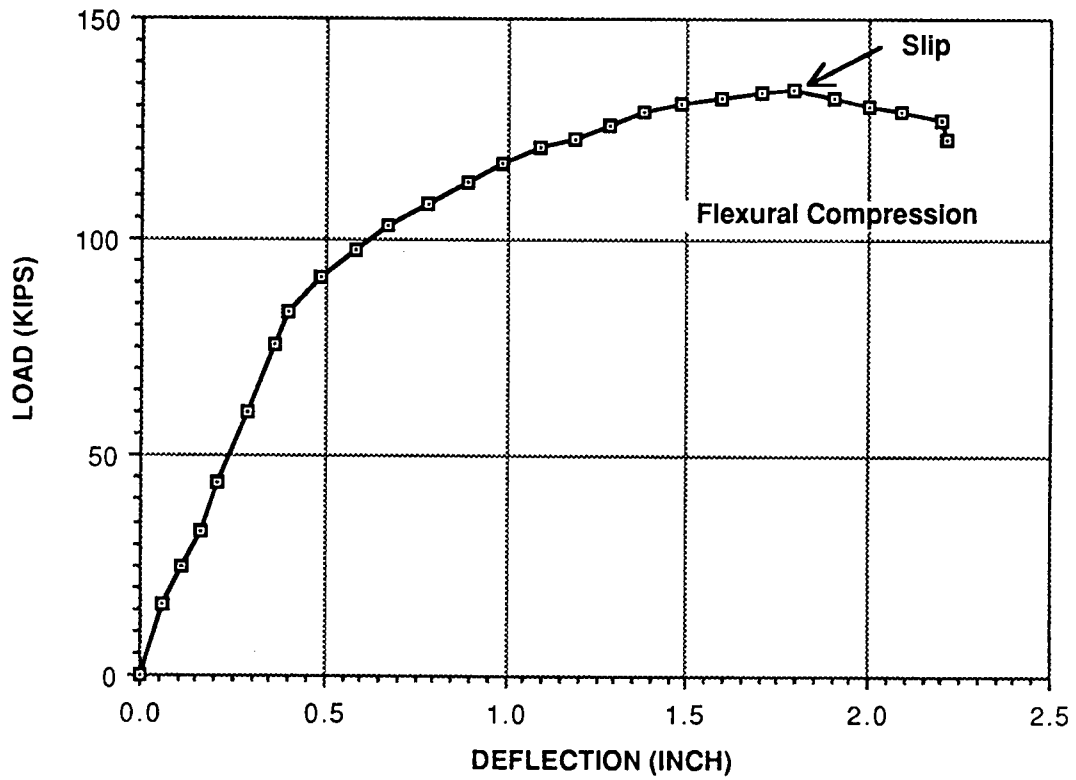
Load Vs. Deflection for Beam 6-2-Ext



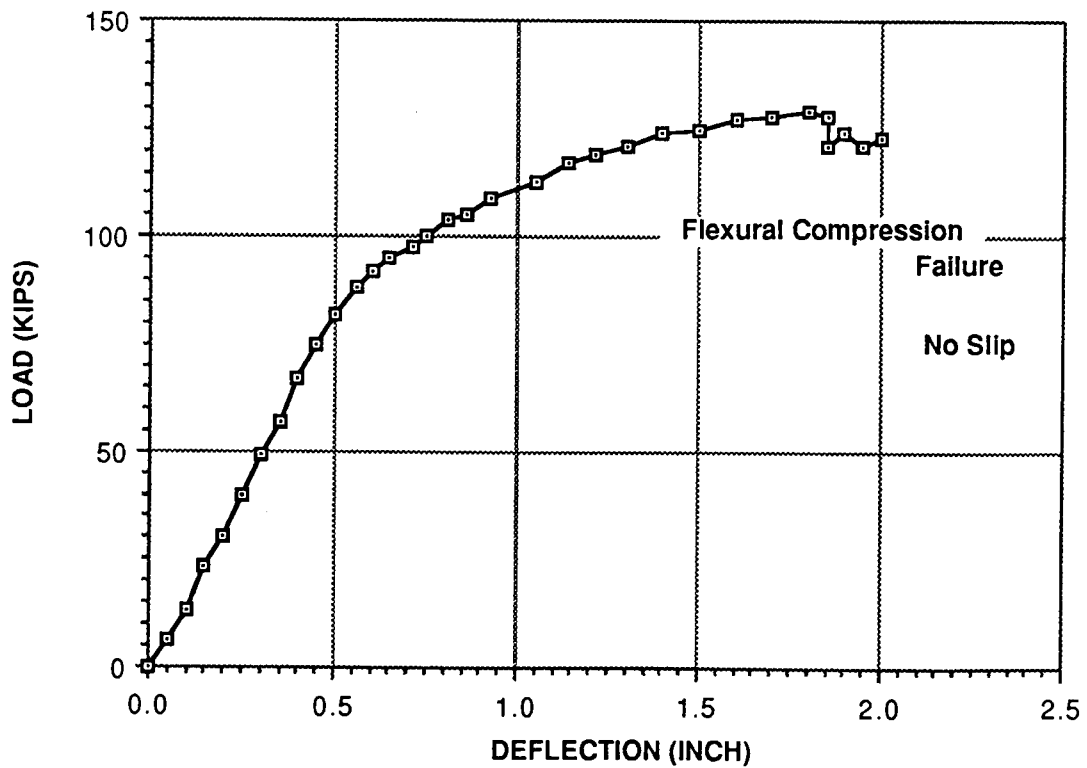
Load Versus Deflection for Beam 6-2-Int



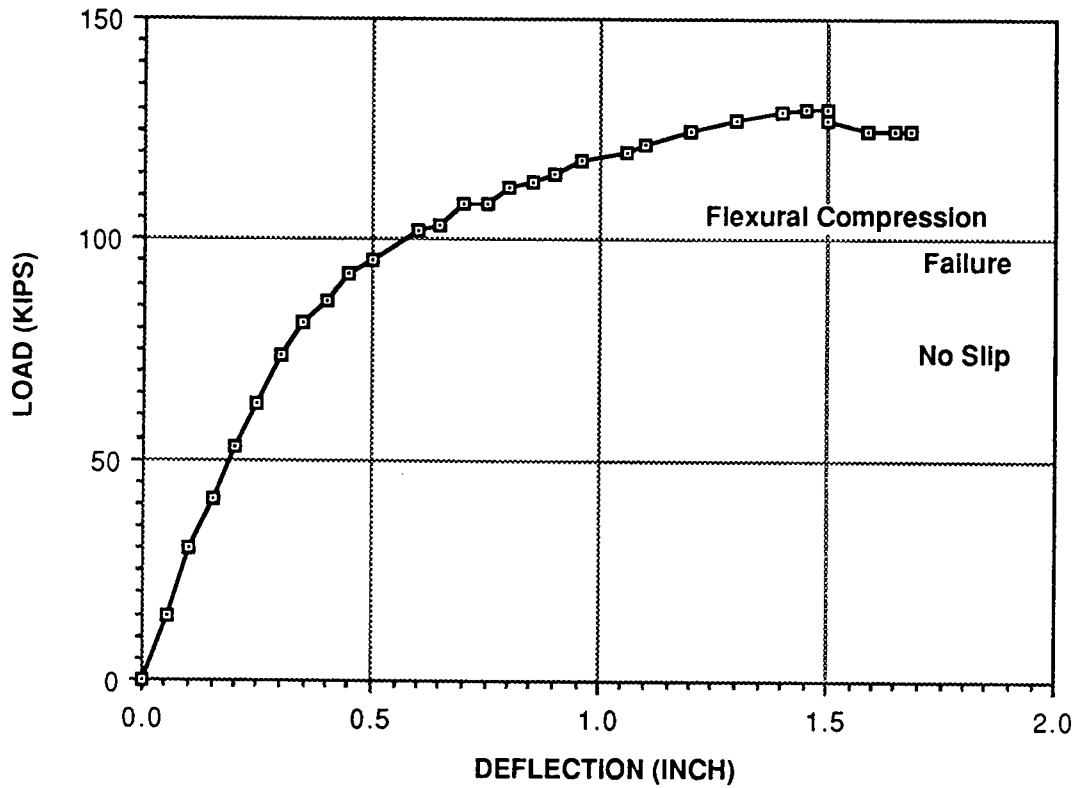
Load Vs. Deflection for Beam 6-3-EXT



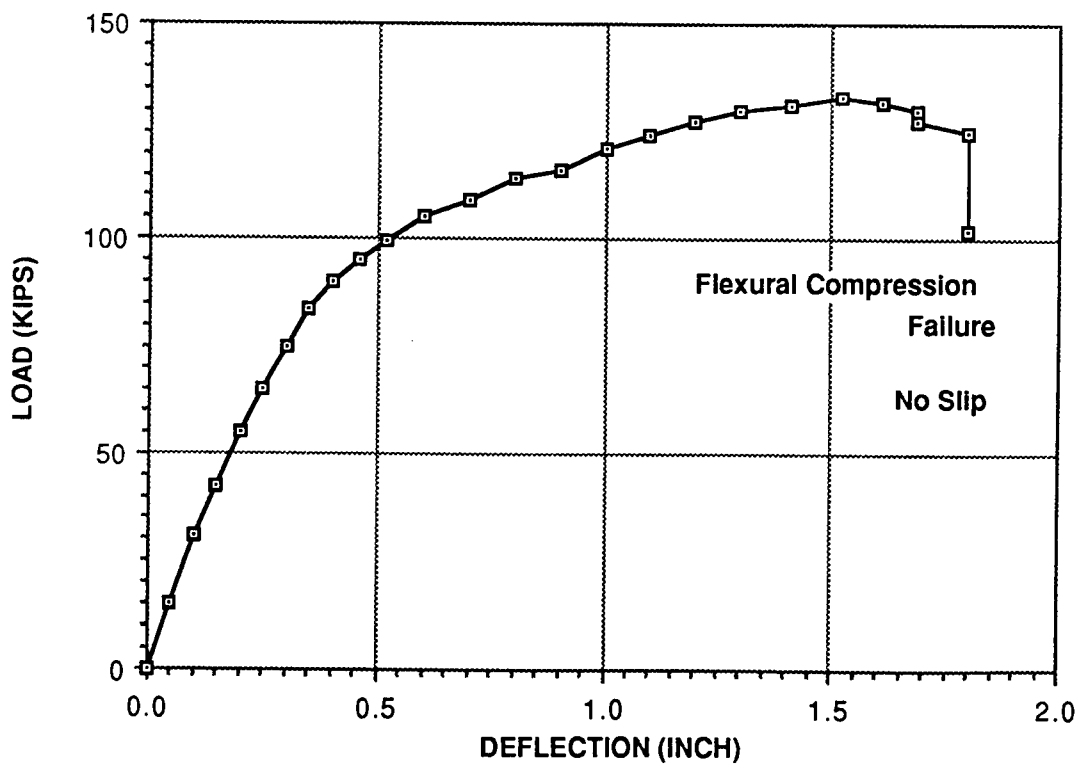
Load Vs. Deflection for Beam 6-3-INT



Load Vs. Deflection for Beam 6-4-EXT



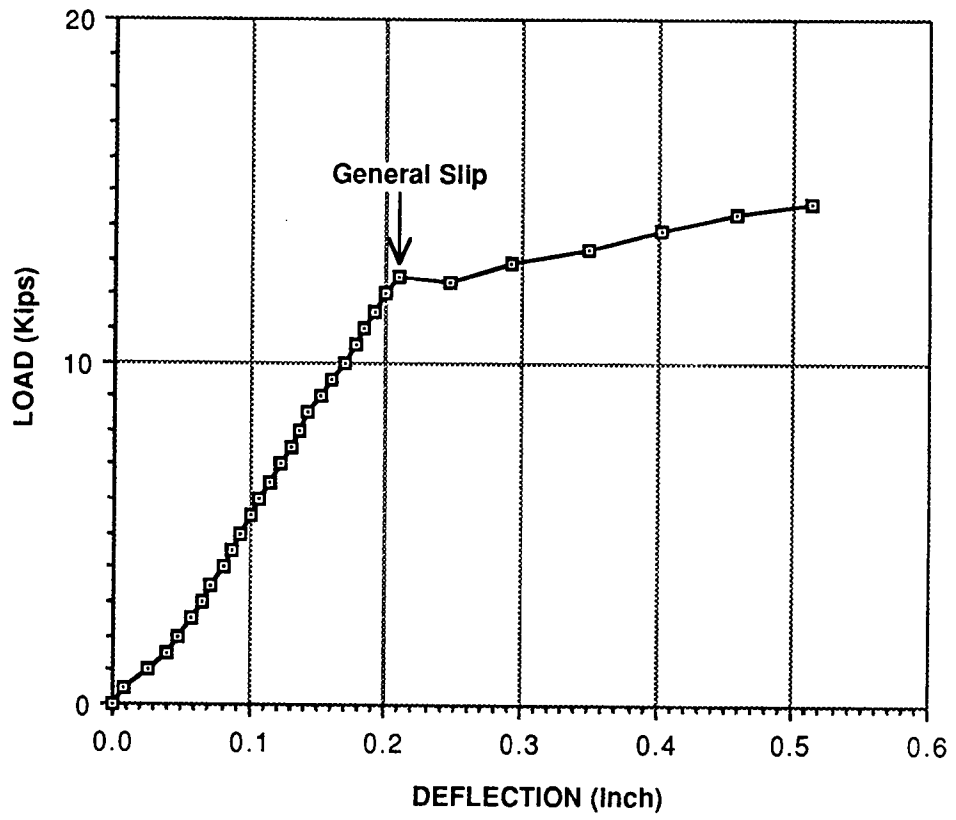
Load Vs. Deflection for Beam 6-4-INT



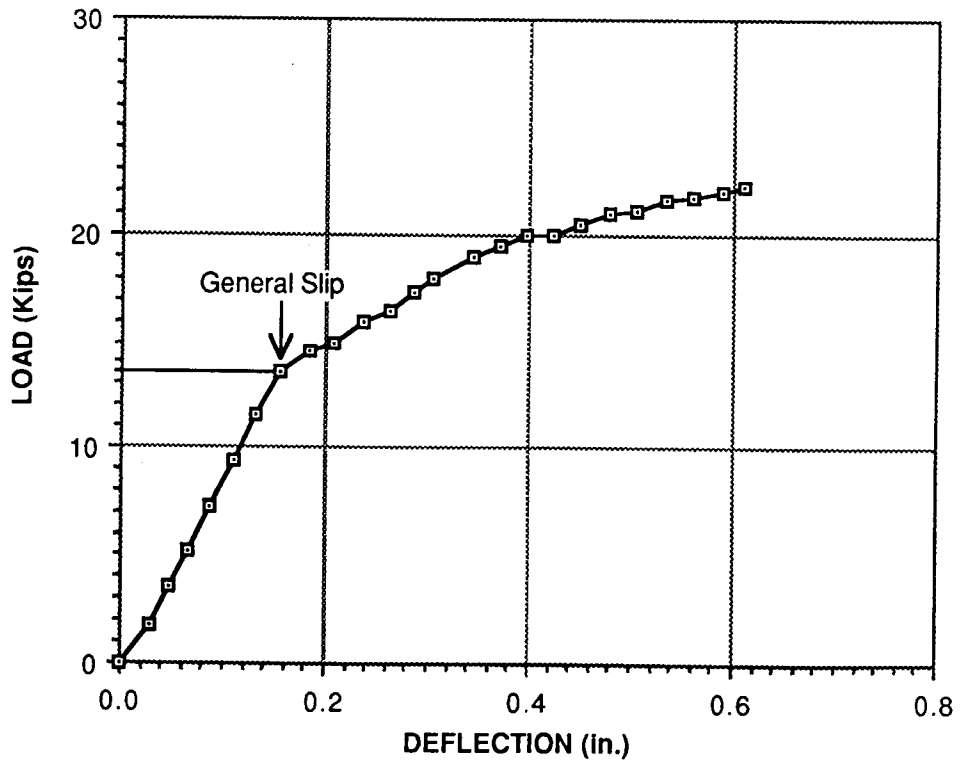
APPENDIX D

**LOAD VERSUS DEFLECTION PLOTS
FOR PULLOUT TESTS**

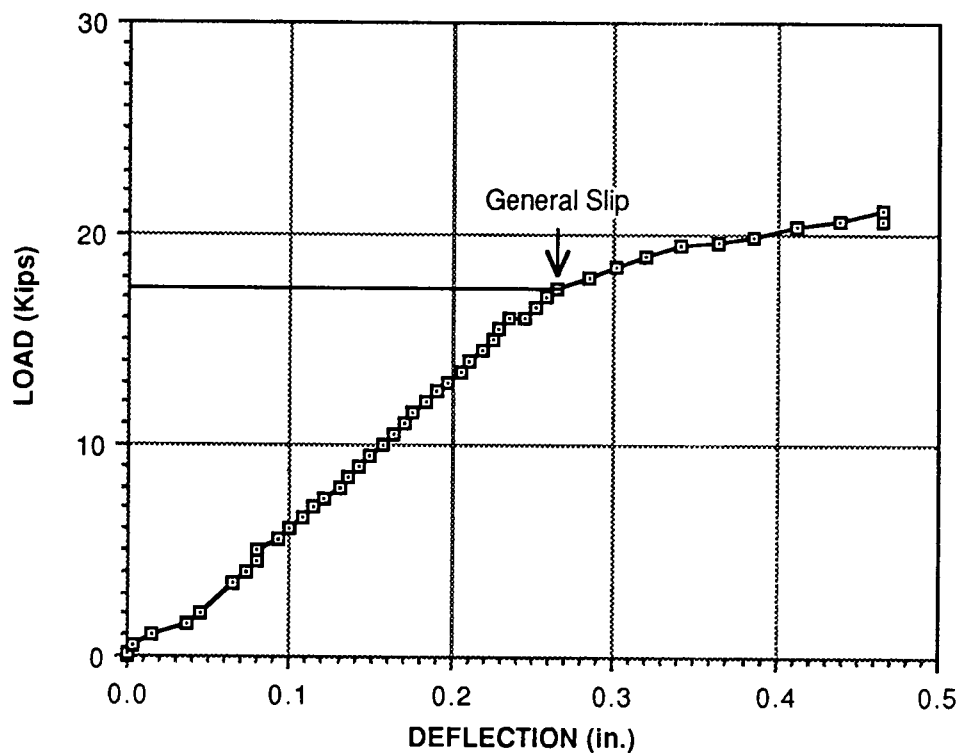
PULL-OUT PRISM 5-1&2-36



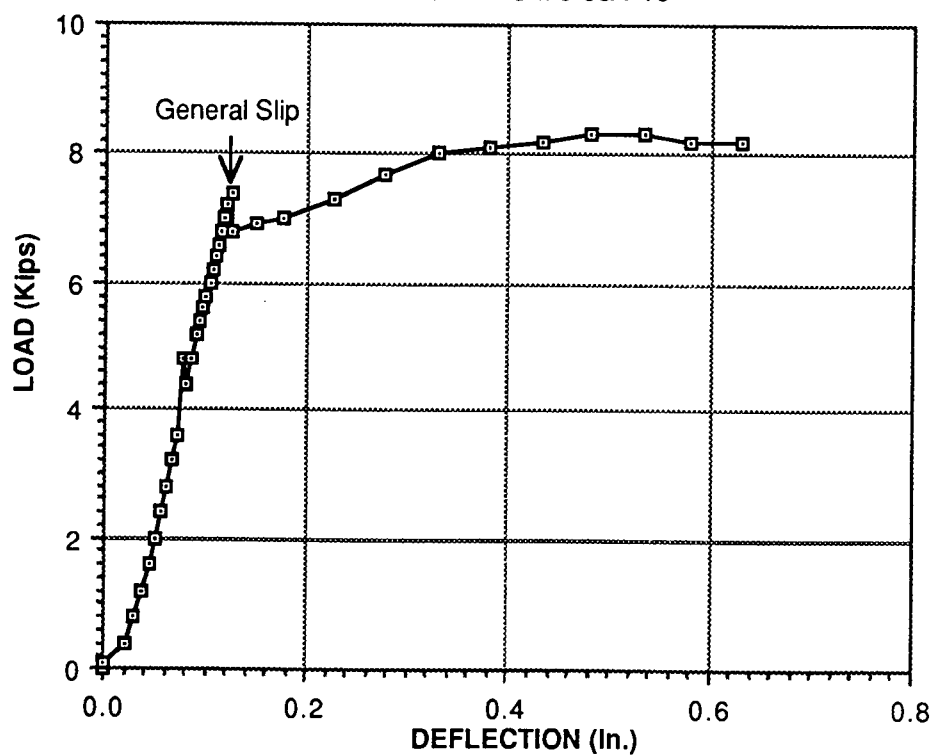
PULL-OUT PRISM 5-FWC-18



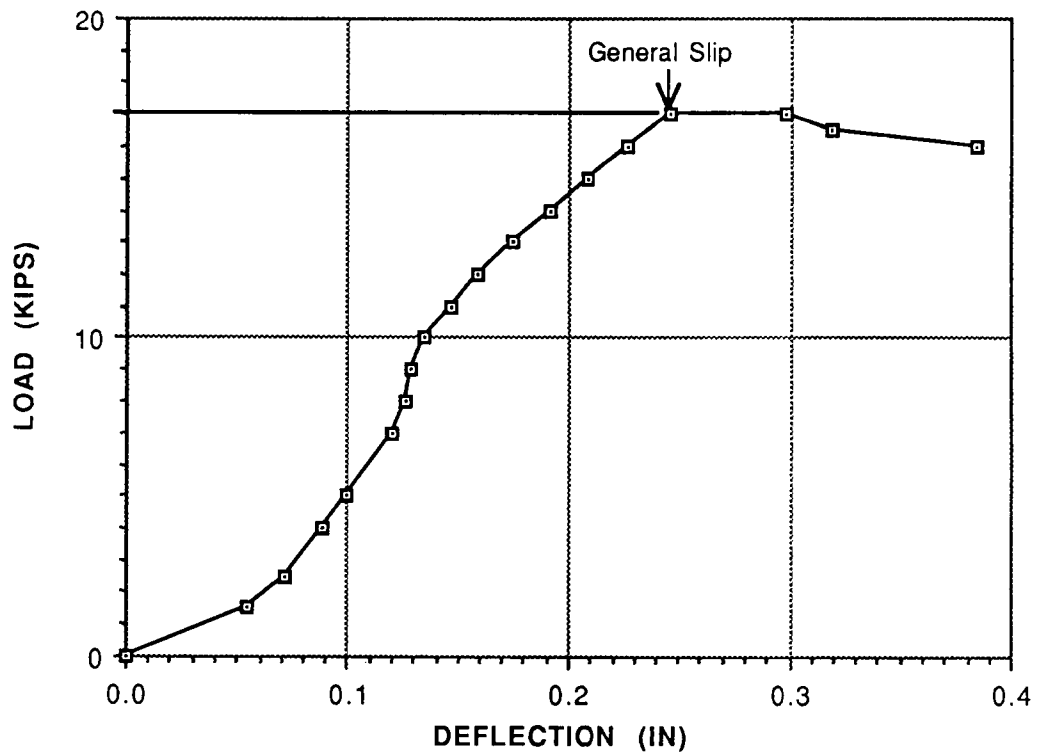
PULL-OUT PRISM 5-3&4-36



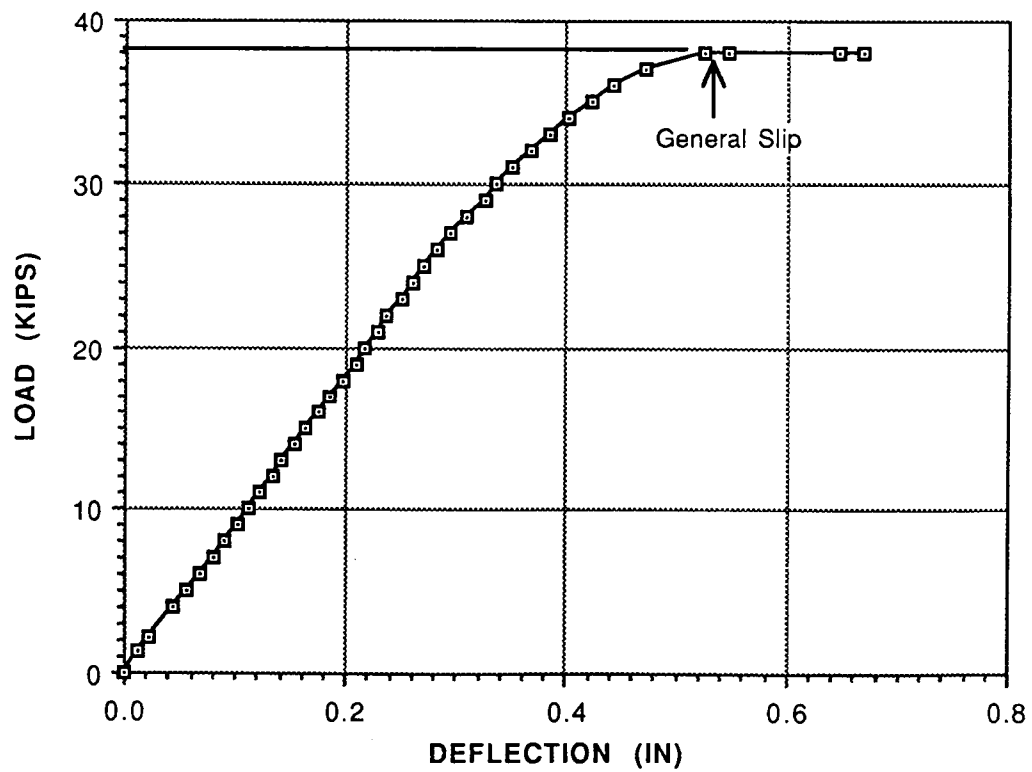
PULL-OUT PRISM 5-3&4-18



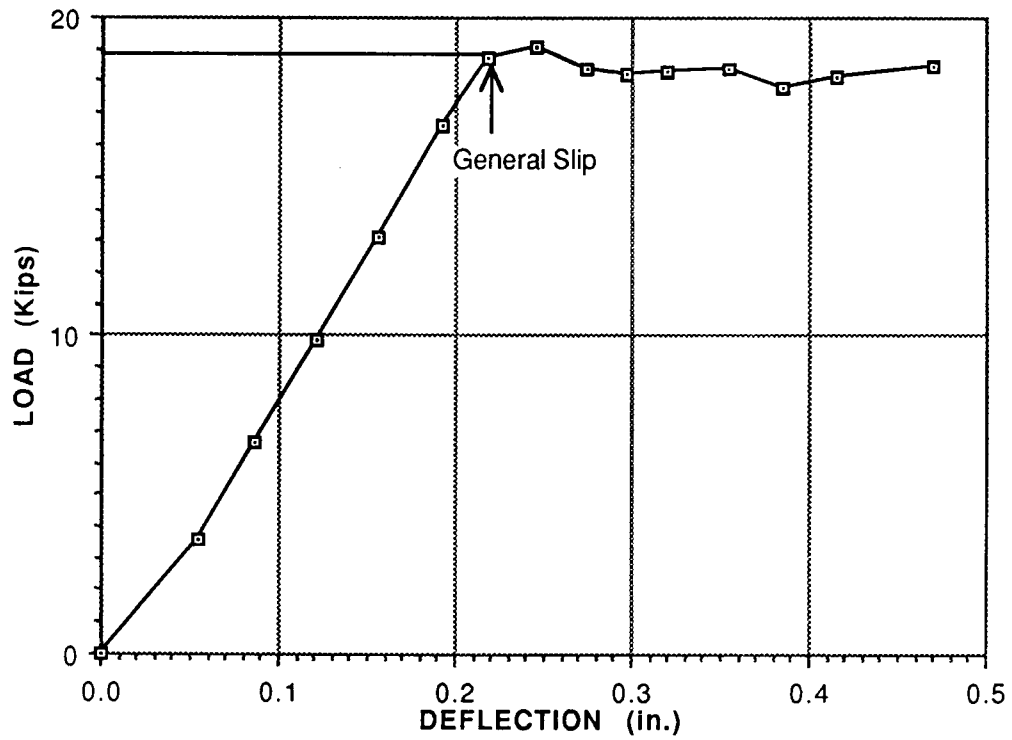
PULL-OUT PRISM 5-SWAI-18



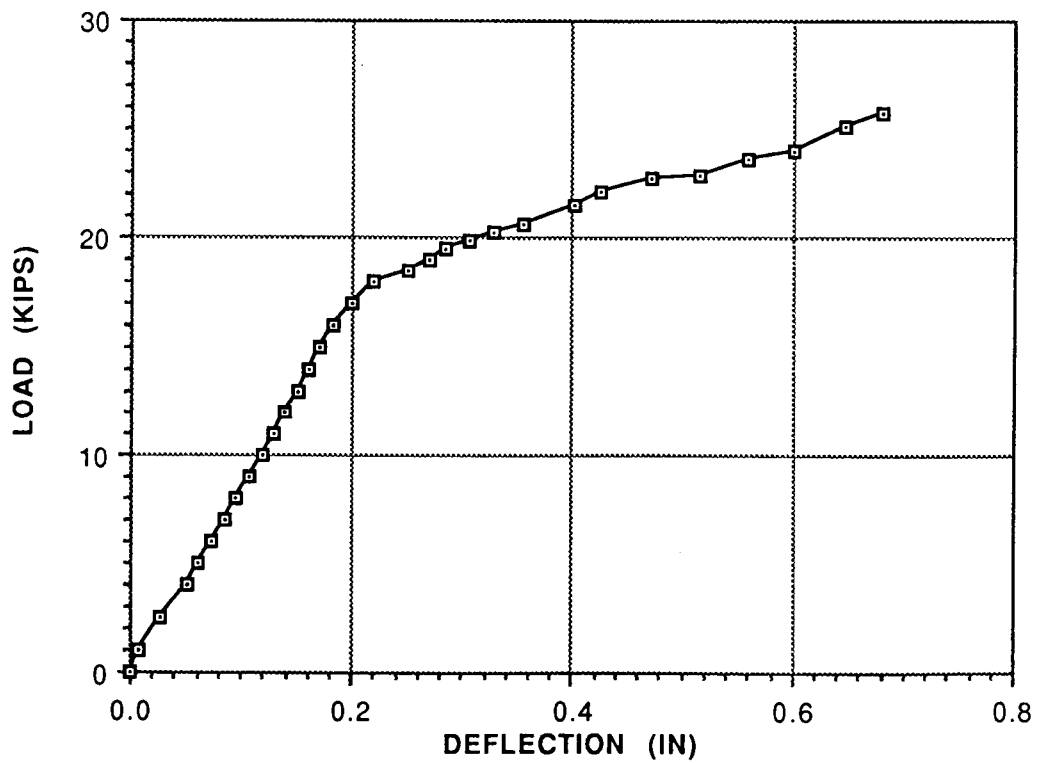
PULL-OUT PRISM 5-SWAI-36



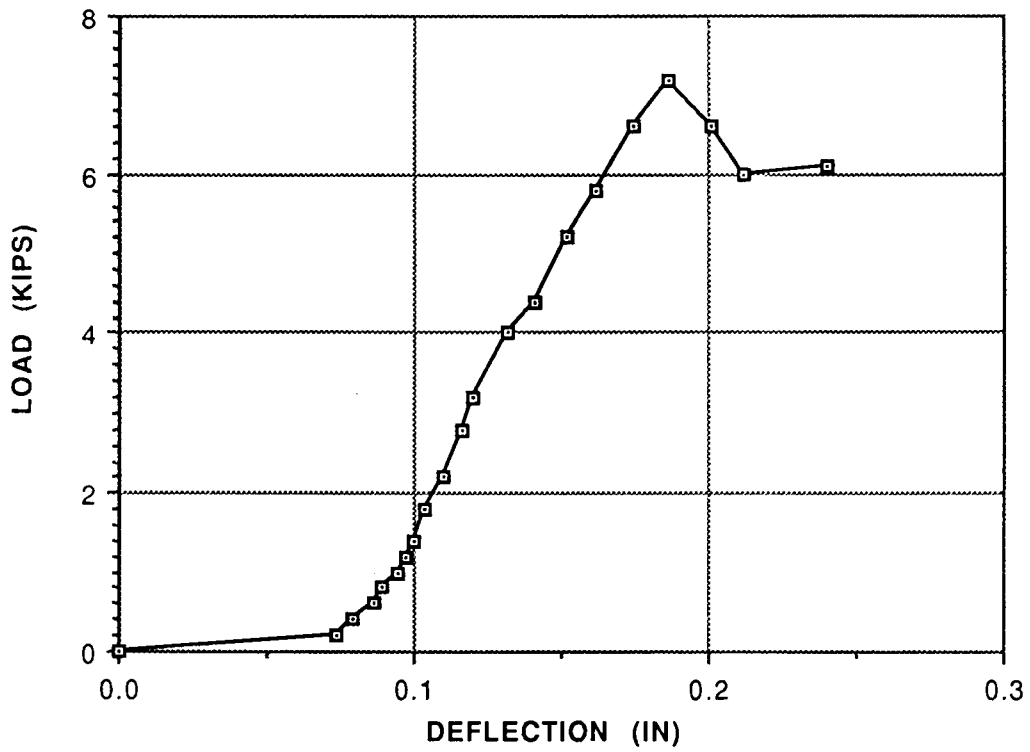
PULL-OUT PRISM 5-ASW-18



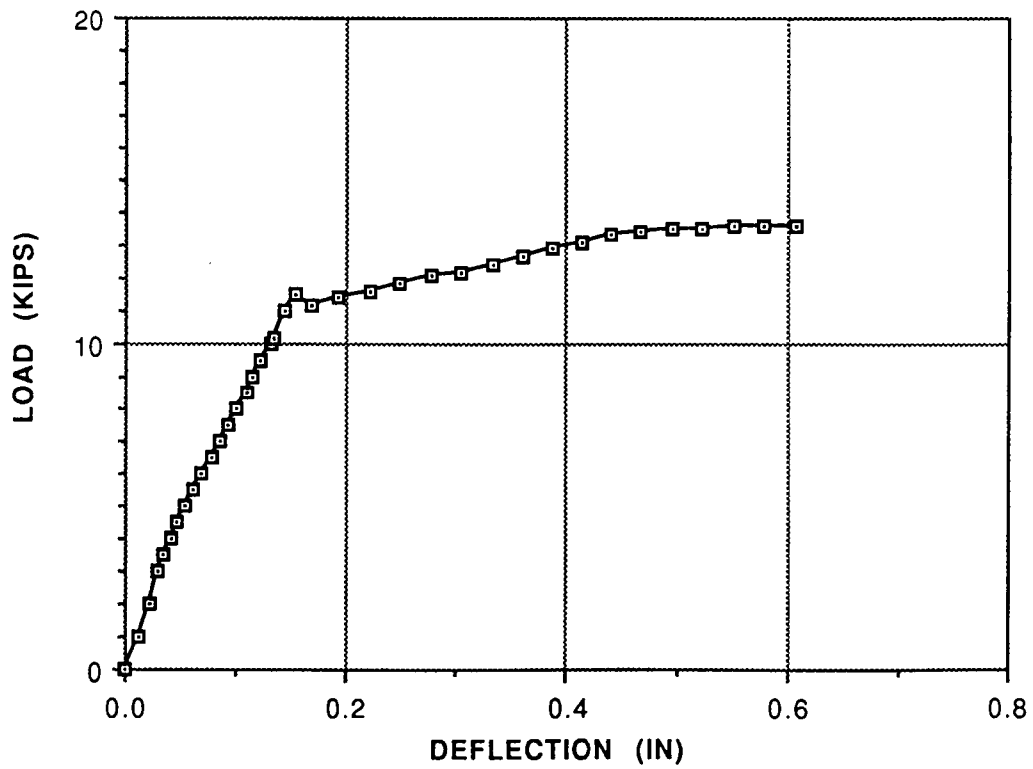
PULL-OUT PRISM 5-UWR-18



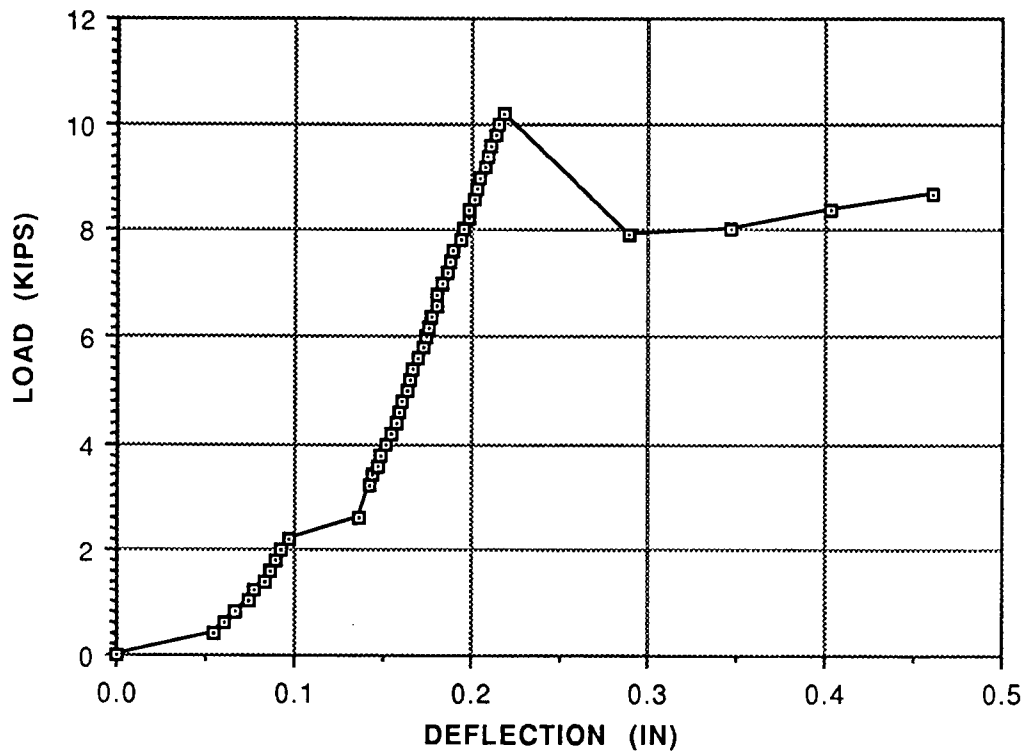
PULL-OUT PRISM 5S-3&4-18



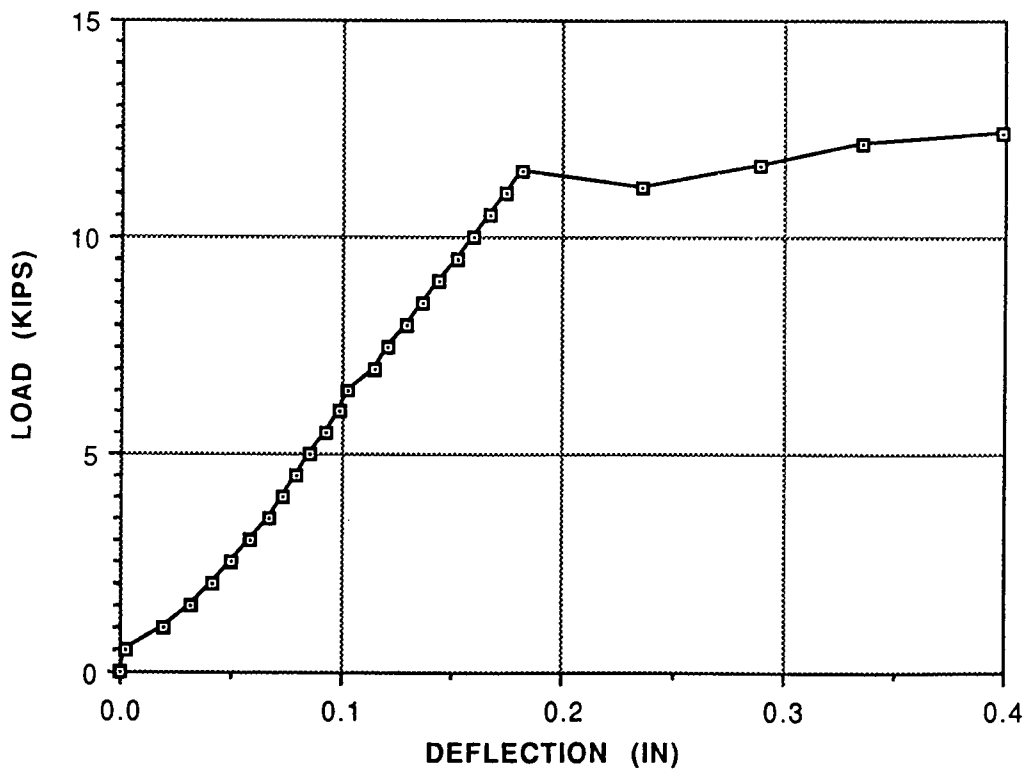
PULL-OUT PRISM 5S-3&4-36



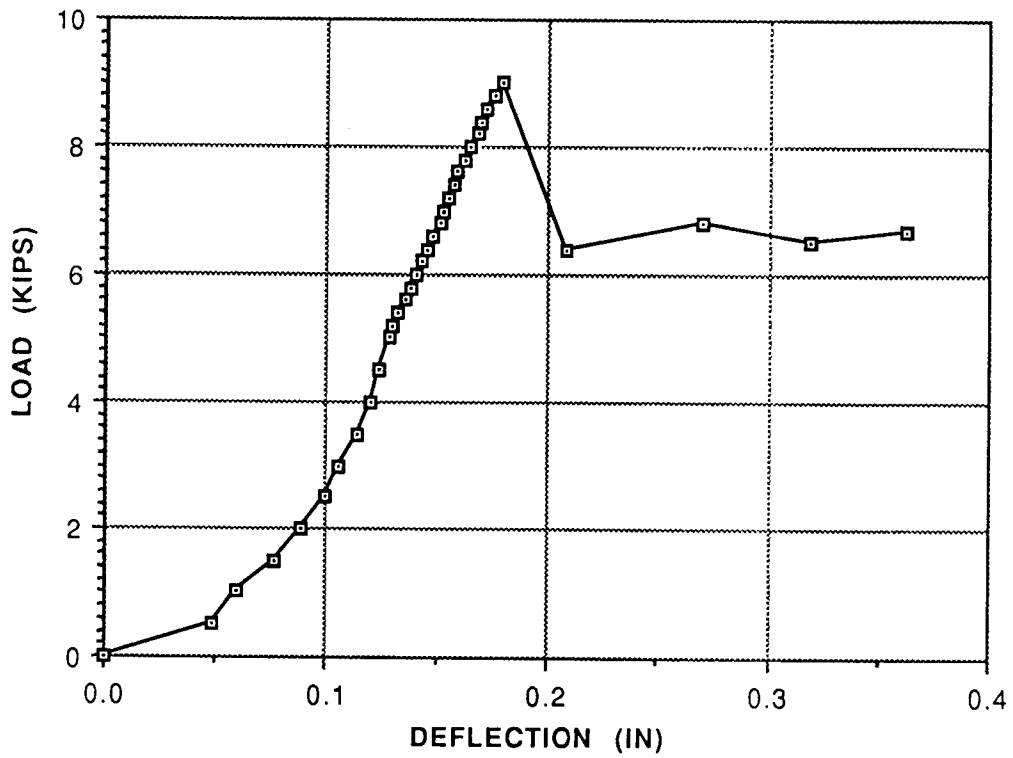
PULL-OUT PRISM 916-1&2-18



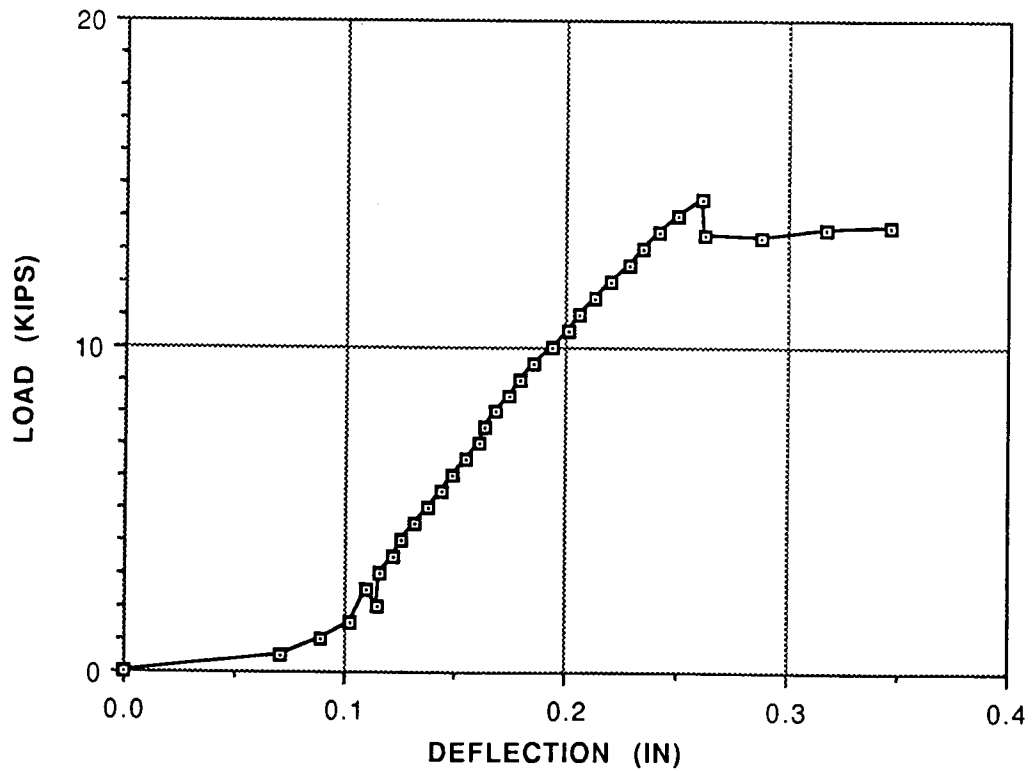
PULL-OUT PRISM 916-1&2-36



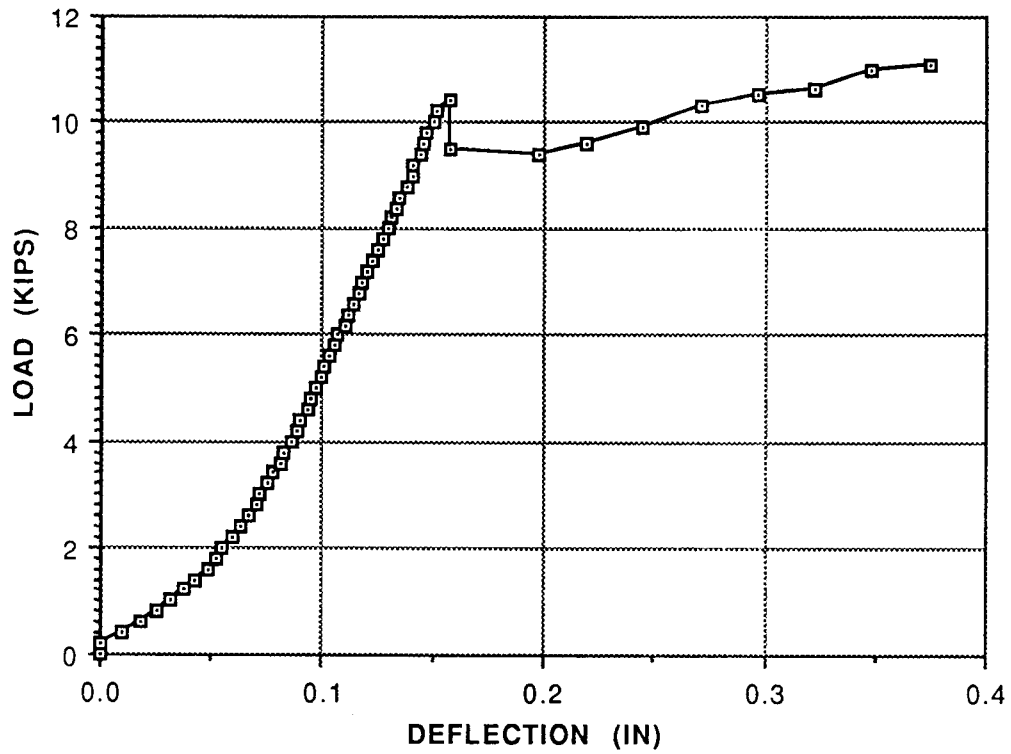
PULL-OUT PRISM 916-3&4-18



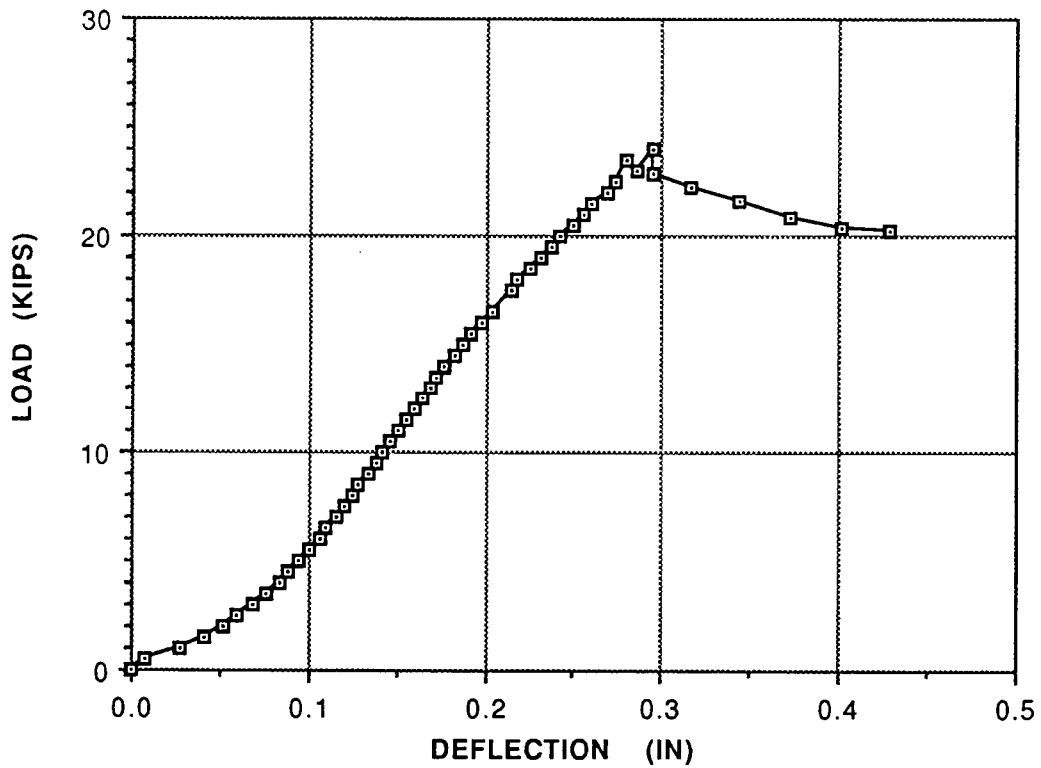
PULL-OUT PRISM 916-3&4-36



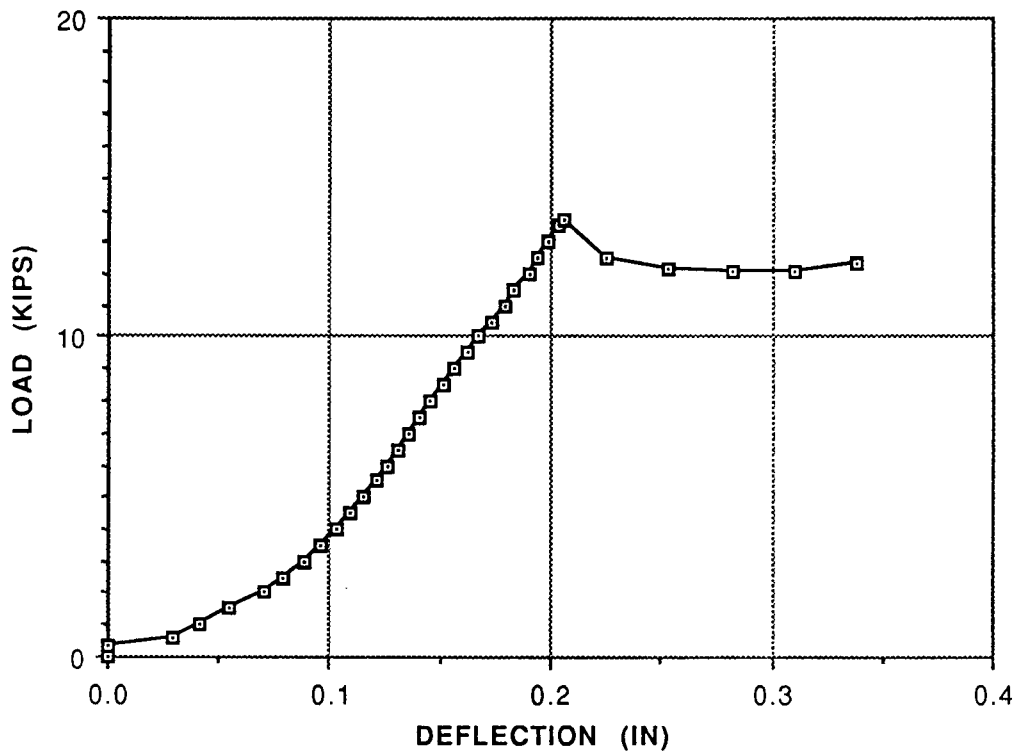
PULL-OUT PRISM 6-1&2-18



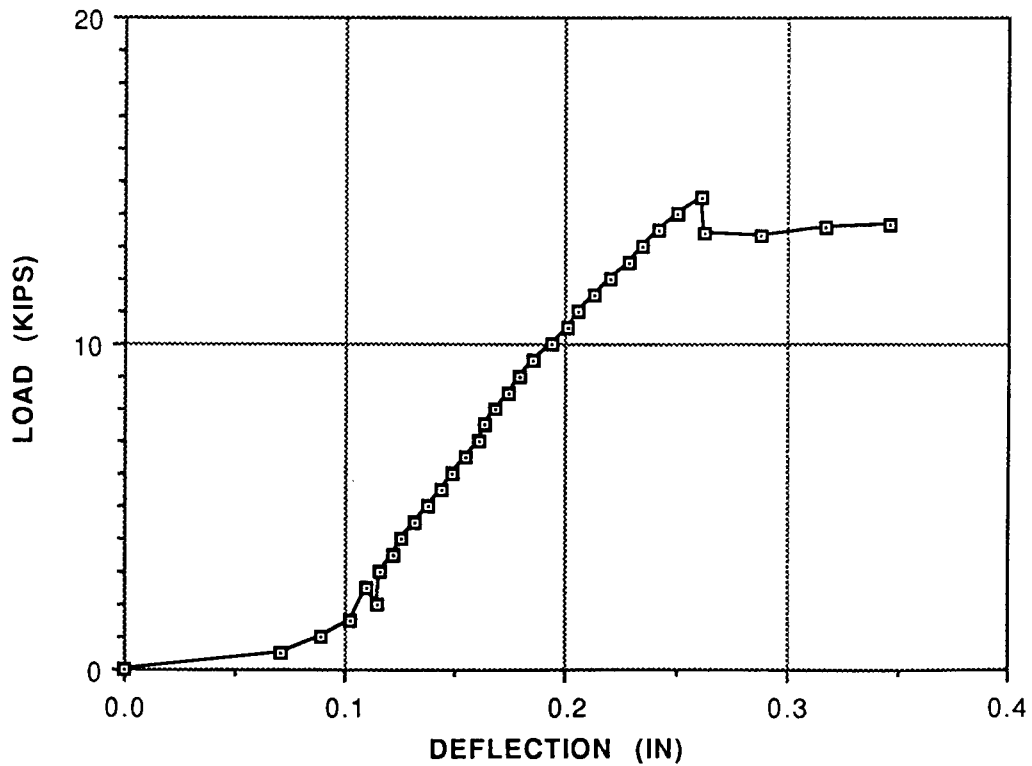
PULL-OUT PRISM 6-1&2-36



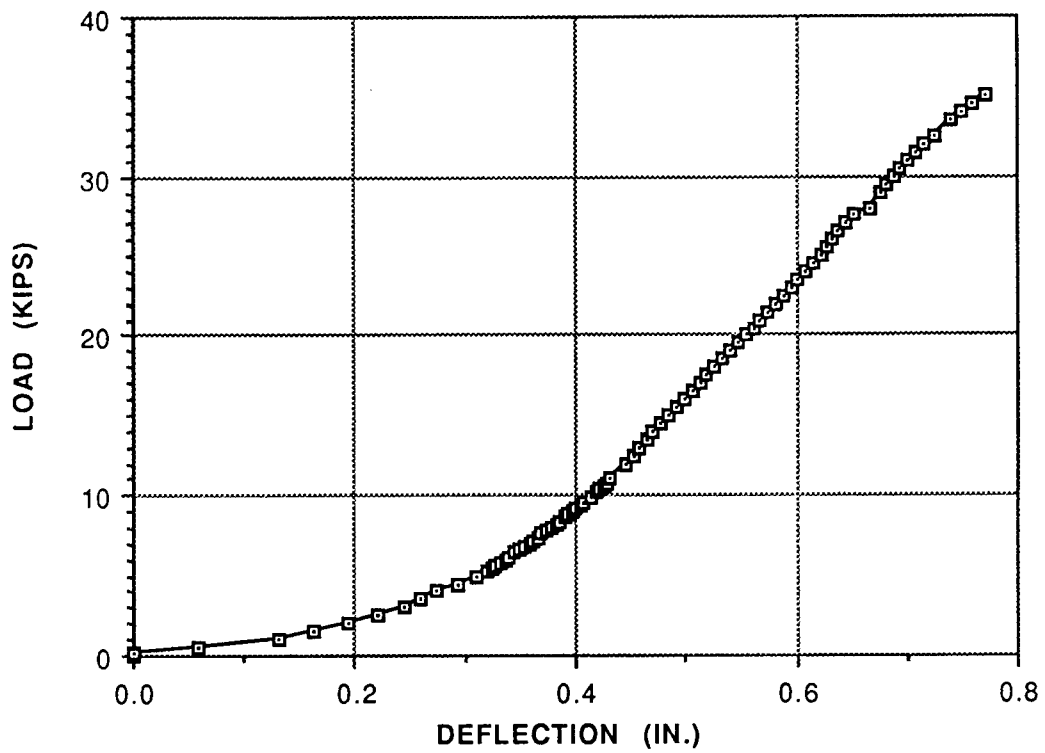
PULL-OUT PRISM 6-3&4-18



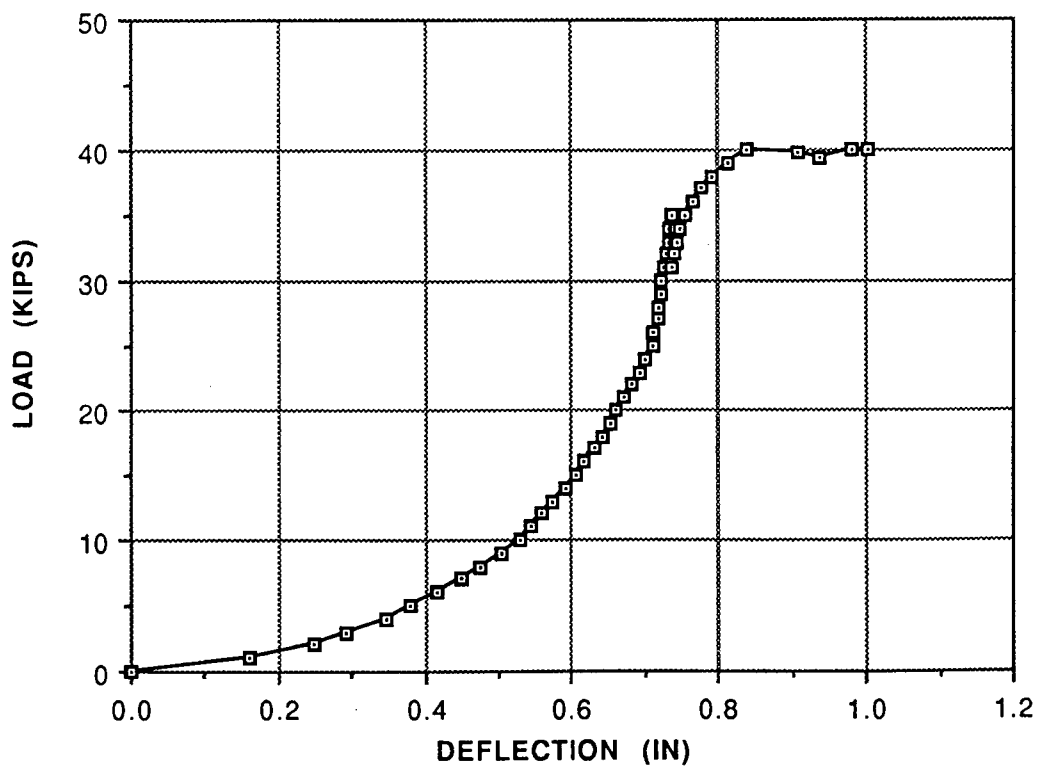
PULL-OUT PRISM 916-3&4-36



PULLOUT TEST 5C-1&2-18



PULL-OUT PRISM 5C-1&2-36



VITA

Chong Key Chew was born in Segamat, Johor, Malaysia on May 3, 1962. He attended a local Chinese elementary school and graduated from Seg Hwa Secondary School in December 1980. He then attended Park Avenue Academy, Toronto, Canada, where he earned an Ontario High School Honours Diploma. He entered the University of Tennessee at Martin and later transferred to the University of Tennessee, Knoxville. Graduated with a Bachelor of Science degree in Civil Engineering with high honors in June 1985, he accepted a teaching assistantship and continued his advanced education at the University of Tennessee, Knoxville. He was awarded a Master of Science degree in Civil Engineering in August 1986.

After having practiced structural engineering for one and a half years with Bender and Associates, a consulting firm in Knoxville, he re-entered The Graduate School of The University of Tennessee, Knoxville. He was a Research Assistant as well as a Teaching Assistant. He was awarded the Doctor of Philosophy degree with a major in Civil Engineering in May 1991.

The author is a member of Chi Epsilon, American Society of Civil Engineers, and a certified Engineer-in-Training.