

Behavior of Larger-Diameter Strands in the Disturbed Region of Prestressed Concrete Girders

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Dedication

I dedicate this dissertation to my wife Sue E. Barnhill Cabage and my family who have supported me with their vigilance and constant perseverance and love particularly over the last four years. Without their support, this study and dissertation would not have been possible.

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I would like to praise God who has been guiding my life in the way that He sees fit for the last 40 years. In this, He has given me the ability, strength and love of learning to accomplish this course of study.

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Abstract

Traditionally, the Tennessee Department of Transportation has been using American Association of State Highway and Transportation Officials – Prestressed/Precast Concrete Institute Bulb Tee (AASHTO-PCI BT) sections for its prestressed concrete bridges. These sections have limited bottom flange widths where the prestressing strands are located. To increase span capacities of these sections, other States have adopted new sections with a wider flange width. Producers in Tennessee have raised concerns about the cost of the new steel forms to accommodate these new sections. As an alternative, this research examines the use of larger diameter strand in conjunction with high-strength concrete and standard AASHTO sections as an innovative and cost effective approach to increase girder capacity to the maximum shipping limit.

Development and transfer lengths for larger diameter strands were found to be much shorter than current models predict. A methodology is proposed for the determination of development length for any size strand. In this study, the effectiveness of casting strand ends within an end diaphragm is demonstrated.

A full-scale bridge beam with larger diameter strand with a composite deck was cast and tested. The behavioral results of this beam were compared to existing models. The existing models were found to accurately predict the behavior of the beam.

This dissertation finds that larger diameter stands can be successfully used in bridge beams. Using these strands will result in reduced bridge costs for spans under 160 feet.

Keywords: Prestressed Girders, Larger-Diameter Strand, Shear Capacity, Bridges, Bond, Disturbed Region, Full-Scale Tests

Table of Contents

1.0 INTRODUCTION	1
1.1 Statement of the Problem	1
1.2 Development Length Literature Review	5
1.2.1 Definition	5
1.2.2 Current Code Provisions	6
1.2.3 ACI Building Code	6
1.2.4 AASHTO Bridge Design Specifications	6
1.2.5 Research on Development Length	7
1.3 End Anchorage	11
1.4 Development of Cracking in the End Zone due to Shear and Bending Stress Combinations	12
1.5 Flexural Member Modeling	15
1.5.1 45 Degree Truss Model and Traditional Approaches	15
1.5.2 Strut-and-Tie Modeling	16
1.5.3 Truss-Arch Model	19
1.5.4 Finite Element Modeling	22
1.6 Crack Development in Reinforced Concrete Beams and Shear Transfer through the Cracked Section	26
1.6.1 Truss Angulation Models	26
1.6.2 Compression Field Theory and the Contribution of Tensile Stress across Concrete Cracks	30
1.6.3 Predicting Response in Shear using Modified Compression Theory	32
1.7 Summary of Larger Diameter Strand Research	33
1.8 Recent Shear Design Approaches in Prestressed Concrete Members	34
 2.0 MODEL FOR STRAND BEHAVIOR IN PRESTRESSED CONCRETE	 37
2.1 Strand Properties	37
2.2 Concrete Properties	39
2.3 Interaction between Strand and Concrete	40
2.3.1 Specimen Fabrication and Test Setup	43
2.3.2 Discussion of Testing Results	47
2.3.2.1 Discussion of the 12-inch Specimens	47
2.3.2.2 Discussion of the 24-inch Specimens	48
2.3.2.3 Discussion of the 36-inch Specimens	50
2.3.3 Analysis of Results	51
2.4 Modeling of Strand Behavior	52
2.4.1 Transfer Length Modeling	52
2.4.2 Development Length Modeling	57

3.0	DEVELOPMENT LENGTH TESTING	60
3.1	Estimation of Development Length using Formulas from Previous Research	62
3.1.1	Beam Capacity Checks	65
3.1.2	Measuring Devices	68
3.1.3	Summary of Testing Procedure	68
3.2	Results of Individual Development Length Tests	76
3.2.1	Specimen A0.70-6	76
3.2.2	Specimen B0.70-6	78
3.2.3	Specimen C0.70-6	81
3.2.4	Specimen D0.70-6	83
3.2.5	Specimen E0.70-6	85
3.2.6	Specimen A0.70-8	87
3.2.7	Specimen B0.70-8	89
3.2.8	Specimen A0.70-10	91
3.2.9	Specimen B0.7-10	93
3.2.10	Specimen A0.62-6	95
3.2.11	Specimen B0.62-6	97
3.2.12	Specimen C0.62-6	97
3.2.13	Specimen A0.62-8	100
3.2.14	Specimen B0.62-8	102
3.2.15	Specimen C0.62-8	103
3.2.16	Specimen A0.62-10	104
3.2.17	Specimen B0.62-10	106
3.3	Summary of Small-Scale Test Results	108
3.4	Comparison of Development Length Modeling and Test Results	110
4.0	FULL-SCALE TESTING	112
4.1	Casting of Composite Deck	112
4.2	Hydraulic Control Systems	119
4.3	Instrumentation	120
4.4	Capacity Predictions using Existing Models	126
4.4.1	Compression Field Analysis	128
4.4.2	Predicting Response using Modified Compression Theory	131
4.4.3	Determine Shear Design Using AASHTO General Procedure	136
4.4.4	Determine Shear Design Using AASHTO Simplified Procedure	139
4.4.5	Analysis of Composite Beam using Response 2000	142
4.5	Testing Procedures and Physical Measurements	149
4.6	Summary of Test Results	151
4.7	Results from Instrumentation Data	164
4.8	Summary of Full-Scale Tests	169
4.8.1	Evaluation of Shear Resistance	169
4.8.2	Moment Capacity and Flexural Cracking Predictions	170
4.8.3	Strand Slip Resistance and Tension-Tie	171
4.8.4	Bearing Pad Design Specification	171
4.8.5	Beam Response using Larger Diameter Strand and High-Strength Concrete	172

4.8.6	<i>Constructability of Beams with Larger Diameter Strand</i>	173
5.0	CONTRIBUTIONS OF THIS WORK	175
REFERENCES		176
APPENDICES		186
Appendix A – Strand Material Certifications and Other Information		187
Appendix B – Concrete Material Information		198
Appendix C – Small-Scale Test Data Sheets		218
Appendix D – Field Notebook		256
Appendix E – Full-Scale Data Sheets		311
VITA		313

List of Tables

Section 1.0

Table 1.1 – Calculation of the Development Length Based on AASHTO 2007	7
Table 1.2 – Different Proposed Equations for Development Length	8

Section 2.0

Table 2.1 – Summary of Pull-out Test Results	46
Table 2.2 – Estimated Transfer Lengths of Strand	56
Table 2.3 – Estimated Development Lengths of Strand	58

Section 3.0

Table 3.1 – Different Proposed Equations for Development Length	62
Table 3.2 – Development Length Estimations	63
Table 3.3 – Beam Capacity Checks—0.62-inch Strand	66
Table 3.4 – Beam Capacity Checks—0.70-inch Strand	67
Table 3.5 – Summary of Small-Scale Testing Results	109

Section 4.0

Table 4.1 – Summary Data for each Beam Section	136
Table 4.2 – Summary Data for each Beam Section	139
Table 4.3 – Summary Data for each Beam Section	141
Table 4.4 – Shear Capacity of Composite Beam	170

List of Figures

Section 1.0

Figure 1.1 – Parametric Study Results.....	3
Figure 1.2 - Definition of Development Length	5
Figure 1.3 – Impact of End Anchorage on Shear Failure Modes	12
Figure 1.4 – Principal Tensile Stress versus Maximum Span Length Using 0.5-inch Strand	13
Figure 1.5 – Principal Tensile Stress versus Maximum Span Length Using 0.6-inch Strand	14
Figure 1.6 – Principal Tensile Stress versus Maximum Span Length Using 0.5-inch Strand	14
Figure 1.7 – Principal Tensile Stress versus Maximum Span Length Using 0.6-inch Strand	15
Figure 1.8 – Diagonal Truss Model (Mitchell and Collins 1991)	16
Figure 1.9 – Typical Strut-and-Tie Models from Shahawy and Chen (1999)	17
Figure 1.10 - Strut-and-tie model from Alshegeir and Ramirez (1992)	17
Figure 1.11 - Pretensioned girder: (a) strand details; (b) cracking; and (c) strut-and-tie model	18
Figure 1.12 – Visualization of Comb-type Cracking Pattern from Kani (1964)	19
Figure 1.13 – Free-body Analysis used by Kani (1964).....	19
Figure 1.14 – Free-body Diagram of Concrete Teeth from Kani (1964).....	20
Figure 1.15 – Specimen D0.70-6, Cabage and Ma 2013, Showing Formation of Thrust Angulation Crack	20
Figure 1.16 – Tied Arch Visualization from Shahawy and Chen (1999)	21
Figure 1.17 - Finite element model of a prestressed concrete I-girder (Vadivelu 2009)	24
Figure 1.18 - Maximum principal stress distribution near the end zone with 0.7-inch strand (Vadivelu 2009)	25
Figure 1.19 - Tests of Simply-Supported T-beams Subjected to Uniformly Distributed Loads. Adapted from Morsch and Collins and Mitchell (1991)	26
Figure 1.20 – Equilibrium Considerations for 45° Truss Model (Mitchell and Collins 1991).....	28
Figure 1.21 - Equilibrium Conditions for Variable-Angle Truss, Mitchell and Collins (1991)	29
Figure 1.22 – Compatibility Conditions for Cracked Web Element Mitchell and Collins (1991)	31
Figure 1.23 – Summary of Property Relationships in Reinforced Concrete Beam Mitchell and Collins (1991)	32

Section 2.0

Figure 2.1 – Photograph of 0.7-inch Strand.....	38
Figure 2.2 – C-S-H and Ettringite Molecules from FWA Website	39
Figure 2.3 – Microscopic Picture of Steel Molecules from Artword.net	40

Figure 2.4 - Test Setup and Free-End Slip Measurement with String Potentiometer and Caliper.....	43
Figure 2.5 - The Location of the DEMEC Gauge Points	44
Figure 2.6 - Comparison of the Force-Slip Curves during the Pull-out Stage (12-inch Specimens)	48
Figure 2.7 - Comparison of the Force-Slip Curves during the Pull-out Stage (24-inch Specimens)	49
Figure 2.8 - Comparison of the Force-Slip Curves during the Pull-out Stage (36-inch Specimens)	50
Figure 2.9 – 0.70-inch Strand Behavior with Varying Pretensioning Force	51
Figure 2.10 – Free-Body Diagram of Strand and Concrete Interaction Model	52
Figure 2.11 – Transfer Strain versus Distance from End of Beam (6,366 psi Concrete)	57
Figure 2.12 - Development Strain versus Concrete Embedment Length (6,366 psi Concrete)	59

Section 3.0

Figure 3.1 – Development Length Specimens	61
Figure 3.2 – String Potentiometer Positioned at the End of the Extended Strand	64
Figure 3.3 – Typical DEMEC Gauge Point Layout.....	65
Figure 3.4 – Typical Gilson Insert Placement for each Beam End	68
Figure 3.5 – Load Frames with MTS Actuators at NTRC.....	69
Figure 3.6 – Positioning of Specimens at NTRC.....	70
Figure 3.7 – Live End of Development Length Testing Specimen	70
Figure 3.8 – Side Bracing Frame and Cribbing	71
Figure 3.9 – Load Point Plating	71
Figure 3.10 – Vertical Displacement String Potentiometer Attachment	72
Figure 3.11 – Strand Slip Monitoring String Potentiometer at Beam Live End.....	73
Figure 3.12 – LabView Chassis used to Record Electronic Instrumentation Measurements	73
Figure 3.13 – Dining Fly used to Cut a Specimen	74
Figure 3.14 – Labeling of Crack Patterns during Testing	75
Figure 3.15 – Crack Pattern Collage after Testing	75
Figure 3.16 – Failure Cracking of Specimen A0.70-6 at Load Point	77
Figure 3.17 - Load versus Deflection at Load Point Specimen A0.70-6.....	77
Figure 3.18 - Steel Strain versus Distance from Live End of Beam, Specimen A0.70-6	78
Figure 3.19 – Failure Cracking of Specimen B0.70-6 at Load Point	79
Figure 3.20 - Load versus Deflection at Load Point Specimen B0.70-6.....	80
Figure 3.21 - Steel Strain versus Distance from Live End of Beam, Specimen B0.70-6	80
Figure 3.22 – Failure Cracking of Specimen C0.70-6 at Load Point	81
Figure 3.23 - Load versus Deflection at Load Point Specimen C0.70-10	82
Figure 3.24 - Steel Strain versus Distance from Live End of Beam, Specimen C0.70-6	82
Figure 3.25 – Failure Mode of Specimen D0.70-6	83
Figure 3.26 - Load versus Deflection Curve for Specimen D0.70-6	84
Figure 3.27 - Steel Strain versus Distance from Live End of Beam, Specimen D0.70-6.....	84

Figure 3.28 – Failure Cracking of Specimen E0.70-6 at Load Point	85
Figure 3.29 - Load versus Deflection at Load Point Specimen E0.70-6	86
Figure 3.30 - Steel Strain versus Distance from Live End of Beam, Specimen E0.70-6	86
Figure 3.31 – Failure Mode of Specimen A0.70-8	87
Figure 3.32 - Load versus Deflection at Load Point, Specimen A0.70-8.....	88
Figure 3.33 – Steel Strain versus Distance from Live End of Beam, Specimen A0.70-8	88
Figure 3.34 – Failure Mode of Specimen B0.70-8	89
Figure 3.35 – Load versus Deflection at Load Point, Specimen B0.70-8	90
Figure 3.36 – Steel Strain versus Distance from Live End of Beam, Specimen B0.70-8	90
Figure 3.37 – Failure Mode of Specimen A0.70-10	91
Figure 3.38 - Load versus Deflection at Load Point, Specimen A0.70-10	92
Figure 3.39 – Steel Strain versus Distance from Live End of Beam, Specimen A0.70-10 ..	92
Figure 3.40 – Failure Mode of Specimen B0.70-10	93
Figure 3.41 - Load versus Deflection at Load Point, Specimen B0.70-10	94
Figure 3.42 – Steel Strain versus Distance from Live End of Beam, Specimen B0.70-10 ..	94
Figure 3.43 – Failure Mode of Specimen A0.62-6	95
Figure 3.44 - Load versus Deflection at Load Point, Specimen A0.62-6.....	96
Figure 3.45 – Steel Strain versus Distance from Live End of Beam, Specimen A0.62-6	96
Figure 3.46 – Failure Mode of Specimen C0.62-6	98
Figure 3.47 - Load versus Deflection at Load Point, Specimen C0.62-6	99
Figure 3.48 – Steel Strain versus Distance from Live End of Beam, Specimen C0.62-6	99
Figure 3.49 – Failure Mode of Specimen A0.62-8	100
Figure 3.50 - Load versus Deflection at Load Point, Specimen A0.62-8.....	101
Figure 3.51 – Steel Strain versus Distance from Live End of Beam, Specimen A0.62-8	101
Figure 3.52 - Failure Mode of Specimen B0.62-8.....	102
Figure 3.53 – Steel Strain versus Distance from Live End of Beam, Specimen B0.62-8	103
Figure 3.54 – Steel Strain versus Distance from Live End of Beam, Specimen C0.62-8	103
Figure 3.55 - Load versus Deflection at Load Point, Specimen C0.62-8	104
Figure 3.56 – Failure Mode of Specimen A0.62-10	105
Figure 3.57 – Steel Strain versus Distance from Live End of Beam, Specimen A0.62-10 ..	105
Figure 3.58 - Load versus Deflection at Load Point, Specimen A0.62-10.....	106
Figure 3.59 – Failure Mode of Specimen B0.62-10	107
Figure 3.60 - Load versus Deflection at Load Point, Specimen B0.62-10	107
Figure 3.61 – Steel Strain versus Distance from Live End, Specimen B0.62-10	108
Figure 3.62 – Plot of Testing Results, Strand Development Length versus Concrete Compressive Strength.....	109
Figure 3.63 – Strand Development Length versus Concrete Compressive Strength Model Comparison.....	111

Section 4.0

Figure 4.1 – Close-up of Support Modifications	112
Figure 4.2 – Side Supports.....	113
Figure 4.3 – Construction of Formwork Support Structure Showing Beam Side Supports.....	114
Figure 4.4 – Staircase for Work Access.....	115

Figure 4.5 – Handrails, Composite Deck Form Boards, and Form Preparation	116
Figure 4.6 – Cylinder Preparation and Concrete Property Measurement at Placement	117
Figure 4.7 – Concrete Deck Finishing	117
Figure 4.8 – Concrete Placement using a Yard Bucket	118
Figure 4.9 – Completed Deck Pour.....	118
Figure 4.10 – Series of Actuators to Test Beam Specimens.....	119
Figure 4.11 – String Potentiometer used in Measuring Beam During Testing.....	121
Figure 4.12 – MTS Control Screen.....	121
Figure 4.13 – String Potentiometer used to Measure Horizontal Shear at Deck Interface	122
Figure 4.14 – 150k Actuator with Load Cell Underneath	123
Figure 4.15 – Programmed LabView Front Panel	124
Figure 4.16 – LVDTs used for Strand Slip Monitoring.....	125
Figure 4.17 – Strand Slip Measurement for Second Full-Scale Test.....	126
Figure 4.18 – Beam Properties for Full-Scale Test.....	127
Figure 4.19 – Example of Material and Section Inputs into Response 2000, 4-inch Transverse Shear Spacing.....	142
Figure 4.20 – Shear Control Curve for Varying Reinforcement Spacing.....	143
Figure 4.21 – Predicted Type I Beam Response to Progressive Loading.....	144
Figure 4.22 – DEMEC Gauge Pattern and Measurement between Points	150
Figure 4.23 – Crack Marking at Staged Load Points.....	151
Figure 4.24 – Initial Cracks between 110 kip and 185 kip Load.....	152
Figure 4.25 – Flange Cracks between a Load of 110 kip and 185 kip	153
Figure 4.26 – Colleague of End Zone Cracking	154
Figure 4.27 – AutoCAD Drawing of Test 0.70A Crack Pattern.....	155
Figure 4.28 – Bent Strand within Formwork	156
Figure 4.29 – Web Cracks on Specimen 0.70B at Loading of 285 kips.....	157
Figure 4.30 – Photograph of Damage Beam.....	158
Figure 4.31 – Close up of Damaged Top Flange at Load Point	158
Figure 4.32 – Cracks which Formed at Final Loading	159
Figure 4.33 – Crack Formed by Rotation of End Block	160
Figure 4.34 – Shearing of Bottom Flange at Final Loading, Specimen 0.70B.....	160
Figure 4.35 – Vertical Top Flange Cracks, Specimen 0.70B	161
Figure 4.36 – Crack Formation in Deck under Load Point.....	162
Figure 4.37 – Cracking in the Top Deck, Specimen 0.70B	163
Figure 4.38 – Loading of First Bridge Beam on Truck	164
Figure 4.39 – Load versus Deflection 0.70-inch Strand Beam Tests	165
Figure 4.40 – Concrete Strain Plots at Strand Centroid.....	166
Figure 4.41 – Lateral Movement across Beam/Haunch Interface at 46 inches from End of Beam	167
Figure 4.42 – Strain in Horizontal Shear Reinforcement	168

1.0 INTRODUCTION

This chapter provides an introduction to this research project by providing an economic basis for our research in the first section. Following the economic basis, the remainder of the chapter provides a literature review summary regarding important aspects of this research program.

A major portion of the research was devoted to understanding strand transfer and development lengths and how these apply to larger diameter strand. A summary of previous research is provided in Section 1.2. Full-scale bridge beam testing was conducted to validate the findings of the modeling and small-scale testing program. Part of this validation included an assessment of end zone stresses. End zone cracking has been shown to be a major contributor to strand bond failure by other researchers. Shahawy and Chen (2001) Sections 1.3 and 1.4 provide a literature review and results of a parametric study regarding this issue.

A literature summary of flexural and shear capacity modeling for reinforced and prestress concrete beams is presented within Sections 1.5 and 1.6. Another substantial portion of this research was devoted to developing an assessment of whether existing predictive models could provide a basis for designing beams with larger diameter strand.

Sections 1.7 and 1.8 summarize the current status of research using larger diameter strand and current thought regarding prestressed concrete beam design. This background was included as part of our overall evaluation process for our full-scale testing.

1.1 Statement of the Problem

Precast pretensioned girders are often the most economical bridge superstructure component used in the United States (US) for simply-supported spans up to 160 feet. Pretensioned bonded strands are used to provide the flexural capacity of the component. The largest pretensioning force, and accordingly number of strands, needed to resist the external loads is required at only one location in the full span, generally the midspan section. Strands are generally located below the centroid of the component.

When the strands are released, the pretensioning force imposes both compressive and bending stresses within the girder. The imposed stresses are generally larger than the stresses imposed by the self-weight of the girder resulting in an immediate camber upward. The net compressive stress at midspan often determines how much prestressing force can be applied without exceeding the compressive stress limit at prestress release. In turn, the amount of prestress introduced into the component determines how much additional, external load can be applied without exceeding allowable stress limits. In addition, the midspan section is checked for various strength limit states as required by design.

A second critical section occurs near the end of the member. The “end” section does not have the benefit of the offsetting self-weight moment in reducing the high compressive and tensile stresses. Thus, this section must be checked for excessive tensile stresses at the top of the section. If excessive, some of the prestressing force must be reduced to satisfy the compressive and tensile stress limits at prestress force release.

Prestensioned components, such as I-girders and Bulb-Tees, have been widely used in the construction of bridges. The prestressing strand diameters used in these components are predominantly 0.5- and 0.6-inch, utilizing Grade 270 steel. This combination of section geometry and strand has worked well regardless of the particular shape when used with normal strength concrete.

With the development and use of high-strength concrete (HSC), many States have developed sections with a wider bottom flange to allow the placement of more strands in the bottom flange. Increasing the number of strands will allow a more efficient use of the HSC materials and provide a longer span length. Other States have elected to continue using the American Association of State Highway and Transportation Officials (AASHTO) sections rather than requiring the prestressed girder manufacturers in their State to acquire new forms.

Within the last few years, researchers have been investigating the use of larger diameter strands in prestressed concrete components. (Morcous, et al. 2012, Song, et al. 2013) Strands utilizing 0.7-inch diameter, Grade 270 steel, have been used in mining applications, and based upon preliminary findings the 0.70-inch and 0.62-inch, 330ksi strand can be used in highway structural components. These strands are defined as larger diameter. Bridges using these strands are currently being designed and manufactured. One example is the Pacific Street Bridge over I-680 in Omaha, Nebraska. (Morcous, et al. 2012)

Initial studies of the use of the larger diameter strands indicate that they have the ability to introduce almost twice the prestressing force, when compared with 0.5-inch strand and 135% of the prestressing force when compared with 0.6-inch strand. (Song, et al. 2013) This could result in a significant span capacity increase of the current AASHTO sections. Form replacement would not likely be necessary. The improved capacity of the current section also could lead to other structural efficiencies such as wider girder spacing, reduced structural mass for seismic design, and reduced substructure and foundational requirements, and reduction in the number of piers—these benefits reduce costs, reduce environmental impact on waterways, and improve the design efficiency of precast bridge structures.

Figure 1.1 presents the results of a parametric study that was performed during this research. In the study, a Bulb Tee (BT) -72 was chosen and bridges of different spans were designed by varying the number of reinforcing strands used at each section. Two different reinforcement strands are presented, the 0.60-inch, 270 ksi strand and the 0.70-inch, 270 ksi strand. The study showed that the transportation length limit for the State of Tennessee can be successfully attained using the larger diameter strand.

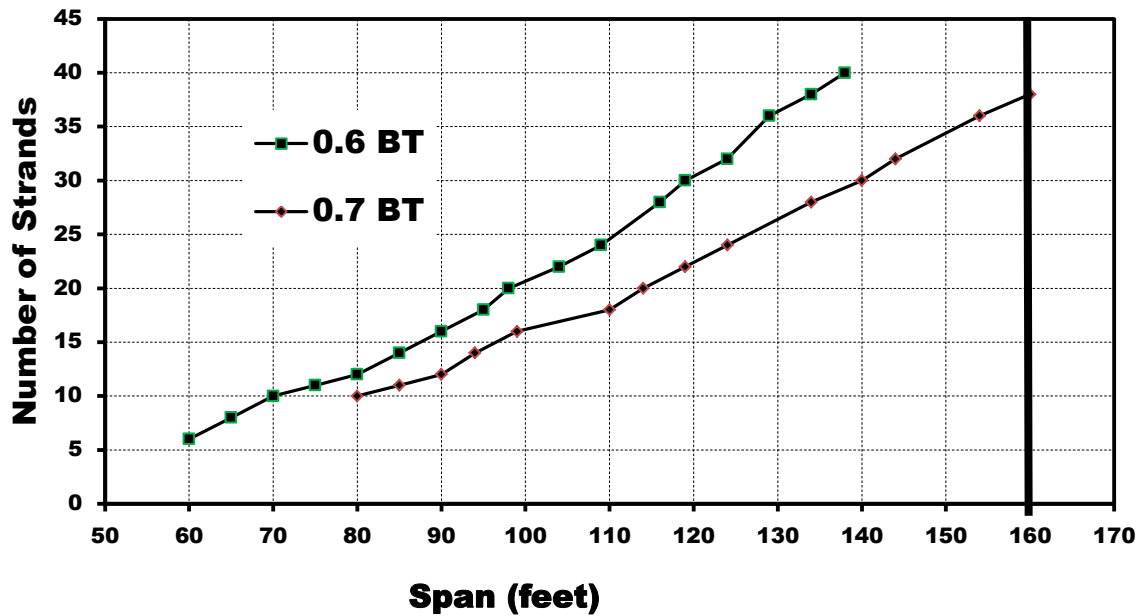


Figure 1.1 – Parametric Study Results

To achieve the enhanced flexural capacity in a concrete girder, several conditions must exist. First, the longitudinal flexural reinforcement steel within the concrete girder must be developed. This development occurs longitudinally along the length of the component due to the bonding of the steel to the concrete. Once bonded, flexural capacity can be attained by the interaction of the longitudinal steel in tension and the compression of the concrete.

The second condition that must be met is that the girder must develop enough shear capacity at the girder end to develop the flexure capacity at the middle of the beam. Shear capacity is attained as a property of the concrete itself and through the use of steel reinforcement within the girder. The combination of shear and moment forces at the girder end can introduce tensile forces which can cause the concrete to crack. When this cracking occurs, the bond between the longitudinal reinforcement and the concrete will become weakened; the longitudinal strands may become debonded and the flexural capacity of the structural girder can become substantially reduced.

Due to its low tensile strength, concrete has a proclivity to crack which can result in an adverse effect in the development of the longitudinal reinforcement. Strand slippage due to cracking within the beam can be prevented in a number of different ways. In 2000, Ma et al. examined extending the strand end into an end diaphragm as a method of end anchorage with 0.5-inch strands in an NU 1100 I-girder section. (Song et al. 2013) A total of five shear tests were performed on two specimens, four of which included end anchorage and one which did not. An

end block was cast at the end of the girder and the longitudinal strands were cast inside this end block to simulate extending strands into an end diaphragm. The three testing specimens with end anchorage failed by the crushing of the web, and one specimen did not fail within the loading limits of the testing apparatus. The specimen without end anchorage failed with a bond-slip mode. Ma's research demonstrated the importance of end anchorage in developing the shear strength of a girder.

The objective of this research is to examine behavior in a prestressed concrete girder using larger diameter strand with high-strength concrete. Differing end anchorage and longitudinal strand positioning will be considered in this investigation. This study provides a further knowledge in shear behavior, evaluates the change in shear capacity using a variety of end conditions, and provides the practicing designer with insight and tools necessary to design girders with larger diameter strand and HSC safely.

This dissertation presents the culmination of a three-year study into the behavior of larger diameter strand and high-strength concrete. The dissertation is divided into two major sections. The first examines the development length of larger diameter strand and provides a new approach and methodology in evaluating the development for all strand types. The second section examines the end-zone behavior of an AASHTO Type I beam constructed with larger diameter strand and high-strength concrete. The following lists the tasks which were performed in this research:

1. Development length for larger diameter strand has been evaluated using single-stranded small-scale specimens. A new development length determination methodology has been developed for reinforcing strand based upon a frictional-based analysis.
2. An investigation into whether the derivation of shear slip resistance and crack spacing as determined by the modified compression theory is appropriate for use in the design of high-strength concrete members with larger diameter strand.
3. An assessment of whether or not the equation for calculating the longitudinal strain at mid-depth is sufficiently accurate when using larger diameter strands.
4. An evaluation of whether the method of determining the angle of principal compression is sufficiently accurate when using larger-diameter strand.
5. An evaluation of whether the assumption that the specification for transverse shear reinforcement can be based upon using the required shear at d_v away for larger-diameter strand.
6. An investigation of the demand and capacity of longitudinal end region reinforcement in member support regions. Also, an examination of how turning up strands within an end block modifies strains observed within the section, particularly at the end zone region.

7. An assessment of the performance of the flexural members at serviceability load levels.
8. An assessment of whether computer-based numerical analysis methods can successfully predict the capacity of test members and then subsequently be used for conducting parametric studies.
9. An evaluation of the ease and reliability of fabricating larger diameter strand bridge members with high-strength concrete by local producers.

This introductory chapter provides a literature review regarding development length and current beam behavioral models. The next two chapters document the small- and large-scale testing programs for this study. The final chapter presents the conclusions and recommendations of this study.

1.2 Development Length Literature Review

1.2.1 Definition

According to the American Concrete Institute (ACI), development length, l_d , is the length of strand required to develop the stress in the strand corresponding to the full flexural strength of the member. The relationship of transfer length, flexural length and development length is shown in **Figure 1.2**.

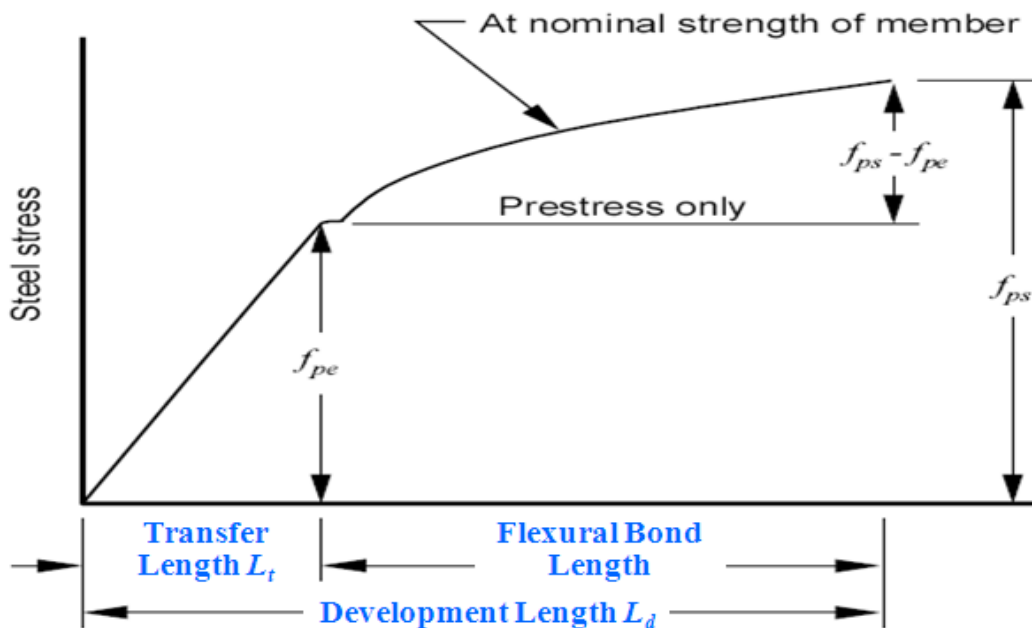


Figure 1.2 - Definition of Development Length

1.2.2 Current Code Provisions

The present provisions for development length of prestressing strand are contained in Section 12.9 of ACI 318-08 (ACI, 2008) and Section 5.11.4 in AASHTO LRFD Specifications 2012 (AASHTO, 2012). Equations for development length were based on tests performed on normal weight concrete members with a minimum cover of 2 in.

1.2.3 ACI Building Code

The provisions in ACI 318-08 read as follows:

Seven-wire strand shall be bonded beyond the critical section, a distance not less than

$$l_d = \frac{f_{se}}{3} d_b + (f_{ps} - f_{se}) d_b \quad (1.1)$$

where,

f_{ps} = average stress in prestressing steel at ultimate load (ksi);

f_{pe} = effective stress in the prestressing steel after losses (ksi); and

d_b = nominal strand diameter (in.)

The first term in the above equation represents the transfer length and the second term represents the flexural bond length. The ACI commentary acknowledges other factors that may affect the development length such as multiple strand configurations, strand surface condition, and the amount of concrete from the exterior edge of the component and the strand.

1.2.4 AASHTO Bridge Design Specifications

In the AASHTO Standard Specifications (1992), the equation for development length is the same as that in the ACI Building Code. In the mid-1980s, Cousins, Johnston, and Zia (1990) measured transfer lengths that exceeded the standard design predictions by a wide margin. Their findings led the Federal Highway Administration (FHWA) to adopt a moratorium on the use of 0.6-inch diameter strands and to increase the development length for other sizes of prestressing strands. The arbitrary 1.6 multiplier from the original FHWA moratorium is incorporated into the AASHTO LRFD Bridge Design Specifications. The development length shall satisfy the following equation:

$$L_d \geq \kappa [f_{ps} - 2/3 f_{pe}] d_b \quad (1.2)$$

where,

d_b = nominal strand diameter (in.)

f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of the member is required (ksi)

f_{pe} = effective stress in the prestressing steel after losses (ksi)

κ = 1.0 for pretensioned panels, piling, and other pretensioned members with a depth of less than or equal to 24 inches; and

κ = 1.6 for pretensioned members with a depth greater than 24

inches.

Based on the current AASHTO Specification, the development lengths of the two full-scale prestressed concrete girder specimens of the project are shown in Table 1.1. This assumes that the equations for development length apply to larger diameter strand as they do for the smaller diameter strand.

Table 1.1 - Calculation of the Development Length Based on AASHTO 2007

Specimen	d_b (in.)	f_{ps} (ksi)	f_{pe} (ksi)	L_d (in.)
1	0.70	270	163.6	180
2	0.62	330	213.3	186

1.2.5 Research on Development Length

In 1988, FHWA issued a memorandum that placed restrictions on the use of seven-wire strands for pretensioned concrete members in highway bridge applications. The FHWA action led to the creation of a large number of research programs intent on measuring the transfer and development length of prestressing strands. Numerous studies have been conducted on both 0.5- and 0.6-inch-diameter strand, often with conflicting conclusions. **Table 1.2** shows the proposed equations by different researchers. Prediction formulas have been developed with no clear consensus among researchers.

It should be emphasized, however, that the impact of variability of development length on bridge beams using 0.5- and 0.6-inch strand is small (PCI 1997). An over-estimation of l_d will not significantly increase the cost of beams from a flexural standpoint, but can have a performance impact in the end regions of the beam.

The following subsections describe major finding of former development length research.

Hanson and Kaar (1959) investigated the flexural bond behavior of pretensioned prestressed beams. The main objective was to gain a full understanding of the flexural bond behavior and use such information to develop design criteria. A theory of bond action was proposed to predict ultimate strength in bond and various factors affecting the bond behavior was studied. The main variables were embedment length and strand size. Also, other variables were studied including percentage of steel reinforcement, reduction of concrete strength, surface condition of the strand, and use of embedded anchorages on bond performance of the strand. This was one of the first studies conducted using strand and established the thought that development length is dependent upon embedment depth.

Martin and Scott (1976) proposed revisions to the ACI 318 development length criteria requirements for prestressing strand. They proposed an approach for short span prestressed members which do not have adequate embedment length to develop full ultimate strength. They reasoned that the existing code equations based on the research conducted by Hanson and Kaar (1959) were unconservative for members with lower steel percentages.

Table 1.2 - Different Proposed Equations for Development Length

Zia and Mostafa (1977)	$l_d = \frac{1.5f_{si}}{f'_{ci}}d_b - 4.6 + 0.18(f_{pu} - f_{se})d_b$
Abdalla et al. (1993)	$l_d = \left[\frac{f_{se}}{3} + 1.7(f_{ps} - f_{se}) \right] d_b$
Mitchell et al. (1993)	$l_d = 0.33f_{pi}d_b\sqrt{\frac{3}{f'_{ci}}} + (f_{ps} - f_{se})d_b\sqrt{\frac{4.5}{f'_c}}$
Burdette et al. (1994)	$l_d = \left[\frac{f_{si}}{3} + 1.5(f_{ps} - f_{se}) \right] d_b$
Buchner (1995)	$l_d = \left[\frac{f_{si}}{3} + \lambda(f_{ps} - f_{se}) \right] d_b$ $1 \leq [\lambda = 0.6 + 40\epsilon_{ps}] \leq 2$
Lane (1998)	$l_d = \left[\frac{f_{si}}{3} + \lambda(f_{ps} - f_{se}) \right] d_b$
Barnes et al. (1999)	$l_d = \frac{5}{4} \left[\frac{f_{si}}{\sqrt{f'_{ci}}} + f_{ps} - f_{se} \right] d_b$
Shahawy (2001)	$l_d = \frac{4f_{si}}{f'_c}d_b + \frac{6.4(f_{ps} - f_{se})}{f'_c}d_b + 10$
Kose and Burkett (2005)	$l_d = \left[95 \frac{f_{si}(1-d_b)^2}{\sqrt{f'_c}} \right] + \left[8 + 400 \frac{(f_{pu} - f_{si})(1-d_b)^2}{\sqrt{f'_c}} \right]$
Ramirez and Russell (2007)	$l_d = \left[\frac{120}{\sqrt{f'_{ci}}} + \frac{225}{\sqrt{f'_c}} \right] d_b \geq 100d_b$

The symbols used in Table 1.2 are defined as follows:

d_b = nominal strand diameter (in.)
 f'_c = concrete compressive strength at 28 days (psi);
 f'_{ci} = concrete compressive strength at time of release (psi);
 f_{ps} = average stress in prestressing steel at ultimate load (ksi);
 f_{pu} = ultimate tensile strength of prestressing strand (ksi);
 f_{se} = effective stress in the prestressing steel after losses (ksi);
 f_{si} = effective stress in prestressed steel after short-term losses (ksi);
 ϵ_{ps} = strain in prestressed reinforcement at nominal strength.

Zia and Mostafa (1977) performed an extensive literature review of past work concerning transfer and development length of pretensioned strands. Zia proposed a development length equation based on his review. This equation took into account the effect of strand size, initial prestress and concrete strength at transfer. The proposed equation gave comparable values of transfer length for the smaller size strands but longer transfer length for the larger size strands, and especially for cases where the concrete strength was low at transfer. They suggested increasing the ACI Code flexural bond length by 25% to ensure that flexural failure at ultimate

stress should occur without any bond slip. This increase was based on their assessment of the research conducted by Hanson and Kaar (1959).

Cousins et al. (1986) studied the effect of epoxy coating on the transfer and development length of prestressing strands. Tested strands had diameters of 0.375, 0.5 and 0.6 inches. The tested strands had either uncoated or epoxy coated surfaces. The research results showed that the three different types of strands require a transfer length of 34, 50, and 56 inches, and a development length of 57, 119, and 132 inches. These values were higher than the estimated transfer and development length by either AASHTO or ACI code equations. These research findings lead to the issuance of the aforementioned FHWA 1988 memorandum regarding the transfer and development length.

Abdalla et al. (1993) conducted research to develop equations for strand development length. The experimental program included the testing of AASHTO bridge girders and box beams. Based on multiple experiments, an equation for the strand development length calculation was proposed.

Russell and Burns (1993) conducted experimental research concerning the strands transfer and development length. The two main objectives of this research were to determine the transfer length and the development length of both 0.5- and 0.6-inch prestressing strands, and to develop design guidelines for the use of debonded strands in pretensioned concrete. This study concluded that the 0.6-inch diameter strand was recommended for use in pretensioned members, without restriction, and subject to the same design provisions of other strand size. They suggested that bond failure initiated by web shear cracking could be prevented by providing both horizontal and vertical shear reinforcement. Flexural cracking and web cracking affected the development length; thus, the loading pattern and cross sectional shape influenced the required development length.

Burdette et al. (1994) conducted a project involving small prisms and full-sized AASHTO Type I girders. This project considered both uncoated and epoxy-coated strands and examined several important variables including strand size and spacing, strand surface condition and stress level at release. Based on the test data, equations for transfer and development length of strand were proposed.

Buckner (1995) presented a summary of research on 0.5-inch strand. He proposed a new expression for transfer and development length after considering all of the research performed since the 1988 FHWA memorandum. Fearing that the recent bond related research on transfer and development length did not strain the strands high enough; Buckner proposed a conservative expression for transfer and development length.

Kilgore (1997) studied the transfer and development length of debonded 0.6-inch diameter prestressing strand in AASHTO Type I concrete beams. He investigated the effect of debonding on the transfer and development lengths of the large diameter seven wire strands placed in concrete beams. The concrete strength ranged from 6.2 to 11.4 ksi. The development lengths were determined indirectly by loading 12 beam ends at different embedment lengths. The test results showed that ACI/AASHTO equations for development length of 0.6-inch seven wire strand with good bond characteristics was conservative.

Lane (1998) conducted an experimental research program to investigate the transfer and development length of prestressing strands. A number of parameters were investigated for possible use in the new transfer length equations. Flexural bond length needed beyond the transfer length to achieve the ultimate strength of the prestressed member was evaluated.

Barnes et al. (1999) conducted research to determine the anchorage behavior of 0.6-inch strands at 2-inch spacing in full-size bridge members. The experimental program consisted of assessing transfer and development lengths in plant-cast AASHTO Type I I-beams. The influence of concrete compressive strengths ranging from 5700 to 14,700 psi was examined. The use of 0.6-inch strand at 2-inch spacing is concluded to be safe. A new equation for development length was proposed for seven-wire prestressing strand.

Shahawy (2001) suggests that for members with depths greater than 24 inches, the AASHTO Standard Specification (STD) equations for development length resulted in an unacceptably low development length while the AASHTO Load and Resistance Factor Design (LRFD) equation yields conservative results. Shahawy also concludes that flexure-shear interaction has a significant effect on the development length of prestressing strands and should be incorporated into the design equations.

Kahn et al. (2002) conducted an experimental research to verify that the transfer and development length of 0.6-inch diameter prestressing strands were less than calculated by the current AASHTO LRFD when high strength concrete is used. The current AASHTO provisions for transfer and development length are recommended for HPC girders with concrete strengths less than 14,500 psi because of their simplicity and accuracy.

In the study conducted by Kose and Burkett (2005), the transfer and development length results from various studies were collected in addition to the results obtained from the tests performed in regional laboratories. Proposed equations for transfer and development length were compared with current equations in ACI, AASHTO, and other codes. Comparisons showed that the proposed equations give better estimates than other equations for 0.5- and 0.6-inch diameter prestressing strand.

Ramirez and Russell (2008) conducted an experimental research to calculate the transfer, development and splice length of strands/reinforcement in high-strength concrete. Transfer and development length of prestressing strand with diameters up to 0.62 inches were studied. The proposed equations correlated the transfer and development length of prestressing strands with initial concrete compressive strength (in case of transfer length), and both initial and final compressive strength (in case of development length). Their testing revealed that the development length requirements diminished with increasing concrete strength.

Floyd et al. (2011) measured the development lengths in self-consolidating concrete (SCC) beams and compared them with those determined from conventional, high-strength concrete beams. Nineteen prestressed rectangular concrete beams were tested to evaluate the development length. They showed that the ACI-AASHTO equation overestimated the development length for all beams by more than 60%.

1.3 End Anchorage

Russell and Burns (1992, 1993, and 1999) performed a series of tests on pretensioned girders with 0.5- and 0.6-inch diameter strands, mainly regarding the anchorage behavior of partially debonded strands. The following conclusions were drawn from the tests:

- Either web shear cracking or flexural cracking in the transfer zones of debonded strands would cause premature bond failure. Therefore, to prevent anchorage failures, girders should be designed so that no concrete cracks will propagate through the transfer zone of a pretensioned strand;
- The potential for cracking was exacerbated due to the reductions of effective prestress force in the zones of debonded strands;
- All debonding should be staggered. Staggering the debond terminations allows for greater precompression in the debond/transfer zones, preventing adverse behavior due to reduced shear capacity or reduced cracking moment. Staggered bond lengths should be determined by the minimum debonding required; and
- If the conditions were met for the anchorage of debonded strands, the rules in the AASHTO Bridge Design Specifications that limit the percentage of debonded strands in the member and in each horizontal row are unnecessary.

Ma et al. (2000) demonstrated that shear strength may be enhanced by extending strands beyond the ends of girders, bending the strands upward, and casting a diaphragm around the extended strands, therefore providing external anchorage. There is significant evidence, as shown in **Figure 1.3**, to support that a girder with end anchorage fails in a web-crushing mode while the one without end anchorage fails in premature shear/bond mode, all other parameters being equal.

Shahawy and Cai (2001) reported tests in which the shear strength of pretensioned girders had been enhanced by external anchors applied to the ends of the girders after the girders were cast and prestress transferred to the concrete.

Lybas et al. (2003) presented the potential of strand end anchorage to enhance shear capacity in the end regions of pretensioned girders. The tests in this study have indicated that the use of external anchorage devices and partial debonding of strand from surrounding concrete represent a promising means of increasing the shear strength contribution of concrete in the prestress transfer region at the ends of girders, effectively converting a pretensioned girder into a post-tensioned girder.



(a) Without End Anchorage



(b) With End Anchorage

Figure 1.3 – Impact of End Anchorage on Shear Failure Modes

1.4 Development of Cracking in the End Zone due to Shear and Bending Stress Combinations

Shear cracking in the Skyway Bridge in Florida attracted a reconsideration of the strand debonding practices in prestressed girder design (Shahawy and Hassan 2009; Illig and White 2010). In the exterior girders of this bridge, AASHTO Type IV girders were used with 61% of the strands being shielded in the bottom flange. The inspection report generated during the investigation of this bridge attributed the cracking to excessive strand debonding at the ends of the girders.

Shear stress cracking at the end of a girder is due to a combination of shear and moment at the end zone. In a parametric study as part of this dissertation, three girders of similar size were examined, the Florida I-Beam 78 (FIB-78), the Texas U-Shape 54 (TU54), and the AASHTO Bulb Tee 72 (BT72). Two different concrete strengths were considered, 7,500 psi and 12,000 psi. At the span lengths demarked in **Figures 1.4** through **1.7**, the primary tensile stress was

calculated at a location 60 times the strand diameter (60 d_b) from the face of the support for each of the three sections. The girders were assumed to be uncracked for simplicity of the analysis. The maximum principal tensile stress results at 60 d_b for each girder versus the span length are presented in the figures. The estimated rupture stress was also plotted.

The preliminary result of this study was to show that during Service III loading conditions, the combination of shear and moment loading can generate tensile stresses above the modulus of rupture. This indicates that cracking may occur at the end zone in these example girders when the concrete strength is below 12,000 psi. One item of note, the primary tensile stress for the BT72 sections did not exceed the rupture strength. This would seem to indicate that this section is not as susceptible to end zone cracking.

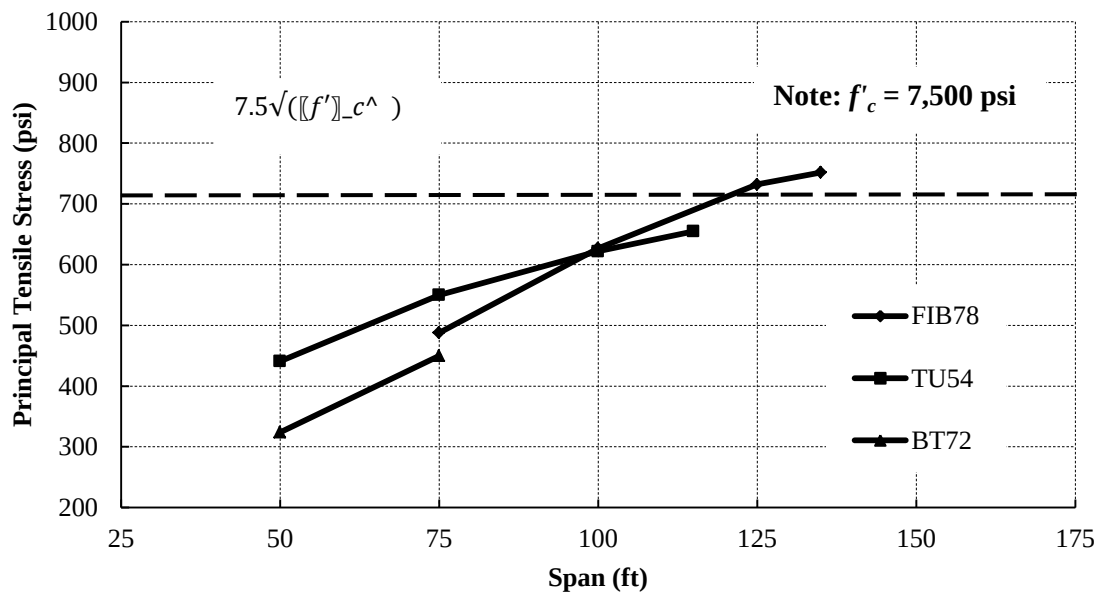


Figure 1.4 – Principal Tensile Stress versus Maximum Span Length using 0.5-inch Strand

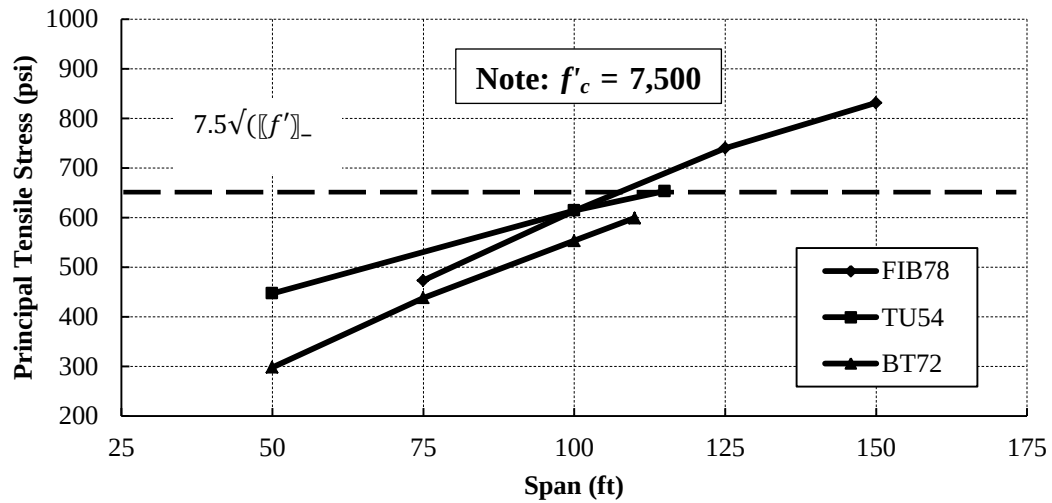


Figure 1.5 – Principal Tensile Stress versus Maximum Span Length using 0.6-inch Strand

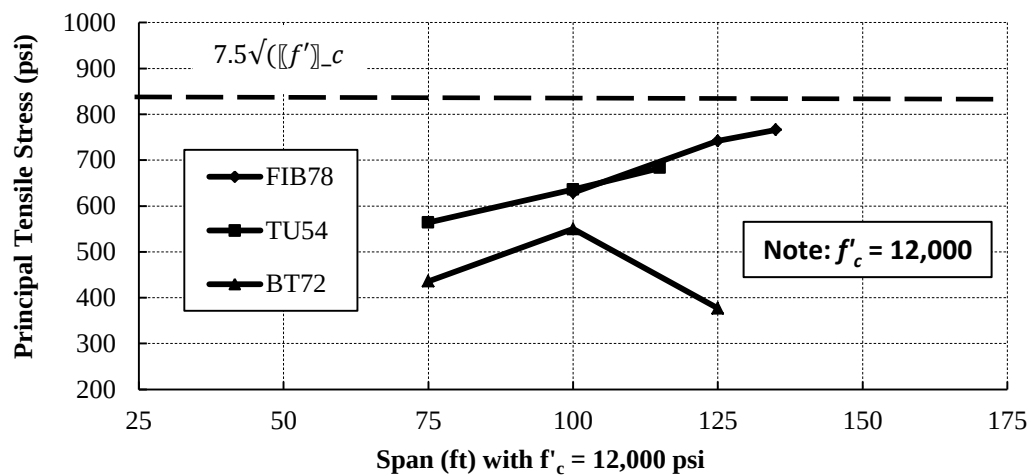


Figure 1.6 – Principal Tensile Stress versus Maximum Span Length using 0.5-inch Strand

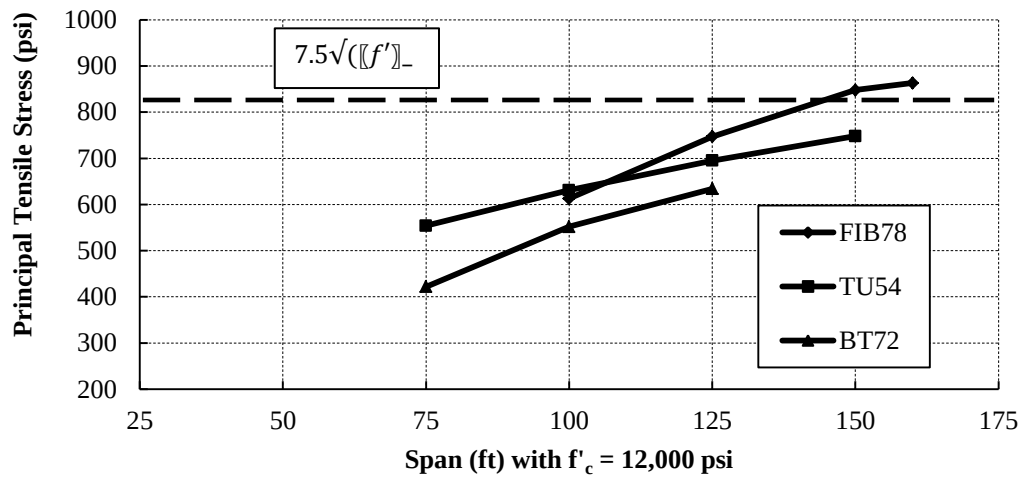


Figure 1.7 – Principal Tensile Stress versus Maximum Span Length using 0.6-inch Strand

1.5 Flexural Member Modeling

1.5.1 45 Degree Truss Model and Traditional Approaches

Modeling of cracked reinforced concrete beams for design has been discussed and debated for over one hundred years. Most models currently being used stem from early research conducted by Morsch where he predicted that shear stress within a reinforced concrete beam would reach its maximum at the neutral axis and would then remain constant from the neutral axis down to the flexural steel (Collins and Mitchell 1991).

One of the original models to explain the flow of forces within a cracked reinforced concrete beam was originated by Ritter in 1899. This is known as the 45 degree truss model. This model envisioned diagonal, compressive members within the cracked concrete beam acting in conjunction with vertical and longitudinal steel reinforcement to form a truss within the concrete beam. Morsch expanded Ritter's work and explained that the compression in the concrete was a compression field rather than a single strut element resisting the shear (Collins and Mitchell 1991).

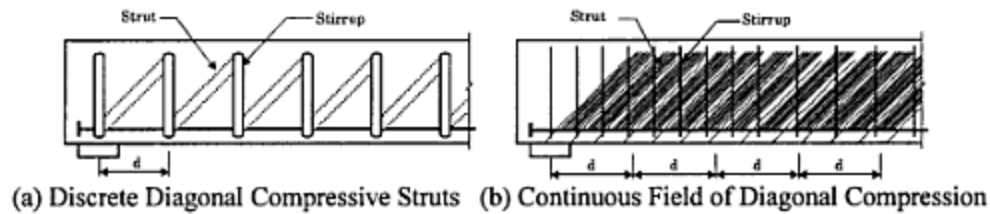


Figure 1.8 – Diagonal Truss Model (Mitchell and Collins 1991)

From this basis, several methods of evaluating force flow and the required flexural and shear reinforcement have developed.

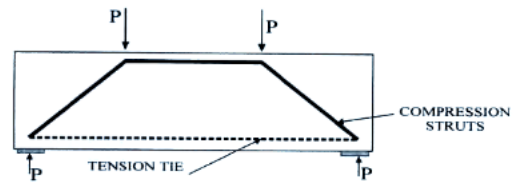
The traditional approach, generally used and recommended by the American Concrete Institute (ACI), is to separate beam capacity into bending and shear components, and then further divide shear into contributions derived from the shear strength of the concrete and the vertical reinforcing steel. This approach is based upon recommendations of ACI Subcommittee 426 and summarized in a report by MacGregor and others (1969) utilizing work conducted by MacGregor, Sozen, and Siess (1965).

1.5.2 Strut-and-Tie Modeling

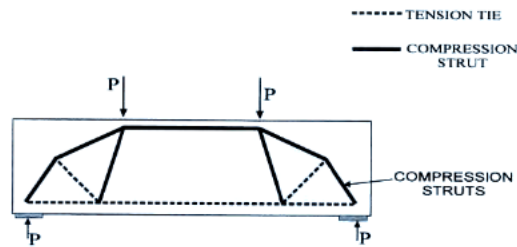
Strut-and-tie modeling is another modeling technique which is popular in Europe and is becoming more familiar in the United States. This method derives its basis from the 45 degree truss model. Strut-and-tie modeling has been the focus of many international researchers, and it is particularly useful in visualizing force flow within a concrete component, especially in “disturbed” regions of a section. Force flow is visualized in the form of a truss and the tensile components, concrete struts, and connection nodes are apportioned based upon the visualized stress at each component.

One of the first authoritative articles on strut-and-tie modeling was presented by Schlaich et al. (1987). In their report to the Prestressed Concrete Institute, the use of strut-and-tie modeling in the design of concrete components, particularly in the disturbed regions of these members, was presented in the form of written examples. They introduced the topic by demonstrating the flow of forces within the disturbed regions and describing how the resistance to the forces can be visualized and then modeled using a truss analogy. The analogy included assigning compression struts to areas where compressive forces occur and tension ties where tensile forces occur. The concrete and steel reinforcement was then designed to counteract the force exhibited by the model. **Figure 1.9** presents an illustration by Shahawy and Chen (1999) showing tension ties and compression struts.

In their report, Schlaich et al. gave several examples of unusual conditions frequently found in beam design. These included beams with designed penetrations, corbels, deep beams, and prestressed and post tensioned girders. In each case, they applied the strut-and-tie model to and presented specific reinforcement detailing for each condition.



a) A Simple Strut and Tie Model



b) A Typical Strut and Tie Model

Figure 1.9 – Typical Strut-and-Tie Models from Shahawy and Chen (1999)

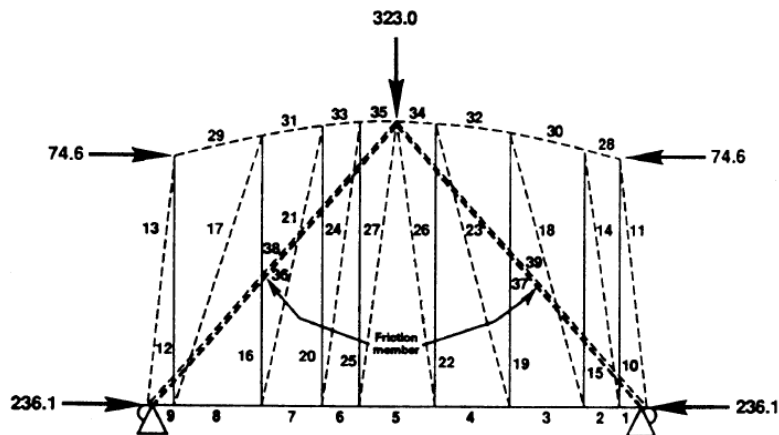


Figure 1.10 - Strut-and-tie model from Alshegeir and Ramirez (1992)

Alshegeir and Ramirez (1992) presented an evaluation of the strength and behavior of prestressed concrete deep girders using the strut-and-tie method. Strut-and-tie system analyses reflecting actual support and loading conditions were developed for three pretensioned deep girders tested to failure. As shown in **Figure 1.10**, the strut-and-tie approach is used to illustrate the effects of prestressing, concrete compressive strength, and reinforcement detailing on the behavior and strength of these members. The strands are represented by a horizontal tie at the centroid of the prestressing steel, and vertical ties are placed at each stirrup location within the

shear span. Particular attention should be placed at points where the prestressing steel is changed, such as debonding points.

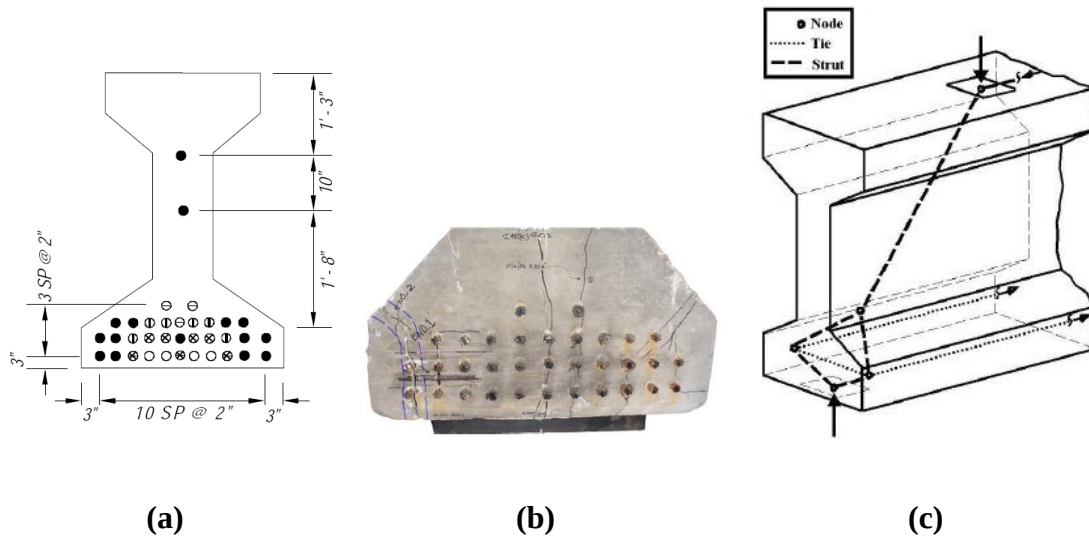


Figure 1.11 - Pretensioned girder: (a) strand details; (b) cracking; and (c) strut-and-tie model

Another design example of strut-and-tie modeling was presented by Wight and Parra-Montesinos (2003). Their example—see **Figure 1.11**—paid particular attention to nodal bearing pressures, nodal bearing areas, and strut widths. The example provided an excellent example on how to split load forces to approximate the strut widths within the system. The report focused on practical detailing in their design example.

Crispino, Cousins, and Robert-Wollmann (2009) developed a design model to guide bridge designers in properly sizing reinforcement to control the length and width of cracks within the end zone regions of prestressed concrete girders. They used strut-and-tie modeling in the prestressed girders in their parametric study. Once the parametric study was completed, they used the developed guidelines and performed full-scale testing on their design recommendations. They found that strut-and-tie modeling can be effectively used in anchorage zone modeling and recommended that the method be adopted by the Virginia Department of Transportation.

Nagle and Kuchma (2007) conducted a shear study of prestressed concrete girders where strut-and-tie modeling was used as an alternate approach in checking the anchorage of longitudinal reinforcement in the beam end zone. The study primarily focused on shear strength and stress of prestressed concrete girders, but they used strut-and-tie modeling to provide an upper bound for the demand on longitudinal reinforcement in both lightly- and heavily-reinforced members. They determined that strut-and-tie modeling is useful in determining the demand in longitudinal reinforcement that does not yield.

1.5.3 Truss-Arch Model

Another approach, which was initially developed by Kani (1964), is to visualize flexural failure in a reinforced concrete beam as a comb with the compressive zone being the backbone of the comb and the tensile zone being the concrete teeth—see **Figure 1.12**. With this visualization, Kani used a free-body method to analyze the mechanism of flexural failure and the mechanism of what he called diagonal failure. He chose the term diagonal failure because of confusion with what constituted a “shear” failure. At the time, some researchers would consider a bond-related failure a shear failure while others would consider them separately. For the purposes of this dissertation, failure mechanisms will be separated into three distinct categories: flexural, bond, and diagonal.



Figure 1.12 – Visualization of Comb-type Cracking Pattern from Kani (1964)

In his work, Kani also developed his understanding—using a free-bodied analysis—of the distribution of tensile forces within the longitudinal reinforcement and how these forces are transferred within the reinforced concrete beam—see **Figures 1.13** and **1.14**. A concept of moment transfer between the “concrete teeth” and the compression zone was developed as part of the overall analysis of his concepts.

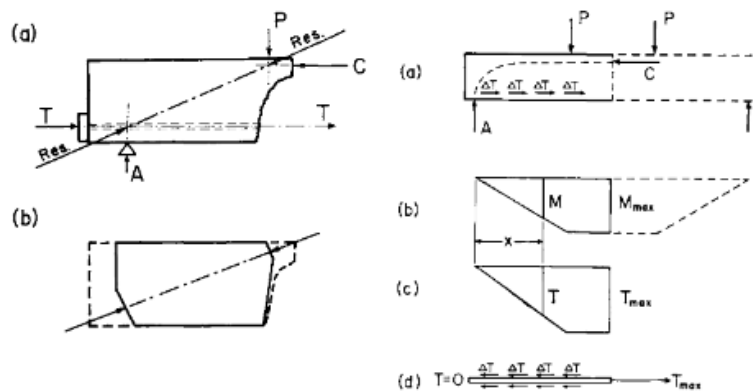


Figure 1.13 – Free-body Analysis used by Kani (1964)

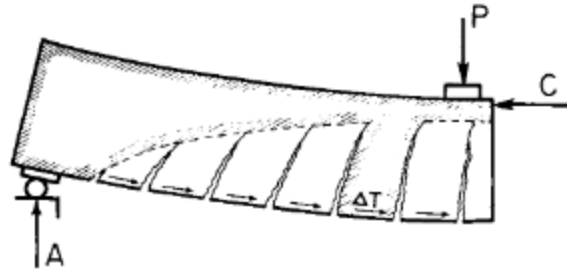


Figure 1.14 – Free-body Diagram of Concrete Teeth from Kani (1964)

In 1999, a similar approach to Kani’s was developed by Shahawy, Batchelor, Chen, and others in cooperation with the Florida Department of Transportation (FDOT) (Shahawy and Chen 1999). In their paper, they pointed out that using strut-and-tie modeling for analysis was subjective and entirely dependent upon the selection of the position of the struts by the designer. Shahawy and Chen showed that assuming a straight-line strut from the load point to the reaction position versus a model that had a variable thrust angle at the reaction site could result in a 50 percent underestimation of shear stress within an end zone region. The occurrence of the thrust angle and the mechanism of its formulation was defined in Kani’s work and can be seen in recent prestressed concrete test results from Cabage and Ma (2013)—see **Figure 1.15**.

Shahawy and Chen (1999) proposed visualizing a prestressed concrete beam as a tied arch with three distinct diagonal crack failure modes to be evaluated and an arch rib evaluation—see **Figure 1.16**. For the end evaluation, a free-body approach was used to quantify the limiting parameter and the capacity of the beam was based upon that evaluation. This approach is useful, but continues to rely upon the selection of a design failure angle and is iterative. The iteration involves balancing the loading with sectional design parameters which includes mild steel shear reinforcement.

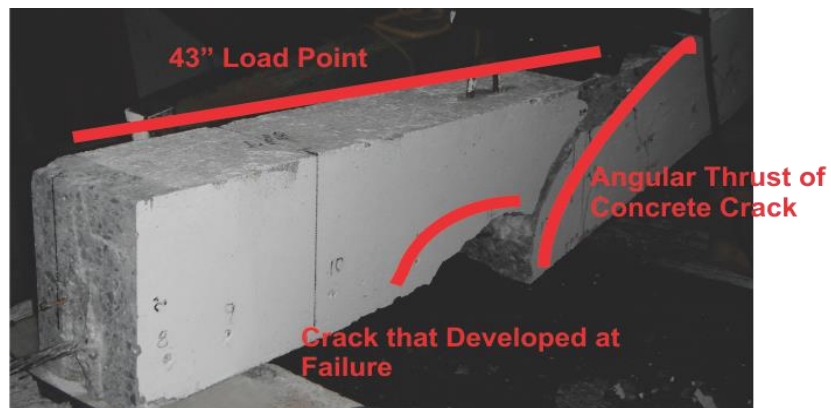


Figure 1.15 – Specimen D0.70-6, Cabage and Ma 2013, Showing Formation of Thrust Angulation Crack

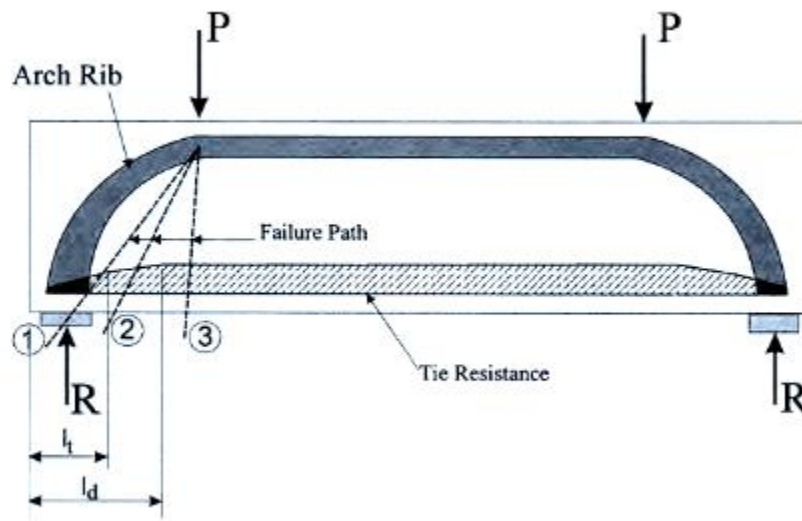


Figure 1.16 – Tied Arch Visualization
From Shahawy and Chen (1999)

Shahawy and Cai (1999) developed a tie-arch model to study the shear behavior of prestressed girder with strand debonding. This model can be used to predict the failure load of prestressed concrete girders and to study the interaction between the tie, shear reinforcement, and compressive struts. The weakness of the longitudinal tie due to excessive strand debonding was considered in the model. In this report, they present three modes of failure in the tied-arch model and correlate the modes of failure to modes of shear failure in the anchorage zone region of a beam. Reinforcement is then specified to prevent these failure modes resulting in the detailing of the beam.

In Shahawy and Cai's report, their design recommendations were compared with existing test data and the ability of the model was tested with these actual results. They found this model to be a rational method of predicting the angle of cracking and the mode of shear failure. Their model was compared to testing data and provided a verifiable response. The proposed method was useful in sizing components without over- and under-sizing reinforcement in prestressed and reinforced concrete members.

In a research conducted by Llanos et al. 2009, AASHTO Type IV test girders were built to replicate existing girders that are in service in Florida. It was found that capacity was not controlled by the typical shear failure mechanisms, but rather was due to the cracking and separation of the bottom bulb flange of the girder. This was a result of the unusual debonding pattern that placed the fully bonded strands out in the bulb flange and the debonded strands under the web. Because of the offset between the strut in the web and the two ties in each flange of the bulb, a transverse tie must be generated.

1.5.4 Finite Element Modeling

The interaction (bond) between concrete and strands is complicated (Stocker and Sozen, 1969). The bond can be attributed to adhesion, mechanical interlock and friction, depending on the extent of bond development and the nature of the strand surface. For the purpose of a better understanding of the bonding/debonding mechanism, the influence of different variables on the stress distribution in strands and surrounding concrete, and the effect of debonding on the performance of prestressed concrete girders, finite element (FE) analysis is commonly conducted.

In the past three decades, finite element models were proposed to capture the response of prestressed concrete members. The FE analysis has evolved from the original one-dimensional model to a three-dimensional model which considers the tri-dimension state of stresses and strains, from perfectly bond to partially bond in which slip between concrete and strands is considered, and from the linear-elastic analysis to non-linear plastic analysis of materials.

To understand the vertical cracks in the end zone of pre-tensioned members during detensioning, Mirza and Tawfik (1978) developed a one-dimensional model based on the assumption of linear-elastic behavior of concrete and steel. They claimed that uncut strands should be lengthened to accommodate the elastic shortening of girders while some of the strands were cut.

Keuser et al. (1983) conducted a two-dimensional FE analysis with the ADINA program. A contact element with a nonlinear bond-slip-relation was introduced into the program. Extensive analytical studies were conducted to investigate the load carrying behavior of prestressed hollow-core slabs as well as the stress state at different load stages. The analytical results gave satisfactory agreement with appropriate test measurements. Keuser and Mehlhorn (1987) developed FE models to investigate the bond behavior in reinforced concrete structures. The superiority of a continuous bond slip function to the bond link element was demonstrated. Consideration of local influences was required by a realistic analytical bond model.

Ahmad and Bangash (1987) developed a three-dimensional finite element analysis incorporating a three-dimensional solid element, an axial line element and a three-dimensional bond-linkage element. The mechanical action of the bond-linkage element was represented by three orthogonal springs connected in the horizontal, vertical and lateral directions to steel and concrete elements. An octagonal prestressed concrete slab was analyzed using the non-linear FE computer program NSARVE, with consideration of three cases: bonded with simulation of the linkage element, perfectly bonded and unbonded. The validity of the specially developed bond-element was established.

Kannel et al. (1997) studied the end cracking of pre-tensioned I-shape girder numerically and experimentally. Three-dimensional FE models were established, in which strands were simulated with truss elements and materials were assumed linear elastic. Also, they evaluated the effect of strand debonding. They concluded that strand cutting order affected the stress distribution in the girder end, and appropriate debonding was beneficial for crack control.

Rabczuk and Eibl (2004) analyzed the performance of prestressed concrete girders under quasi-static loading using a coupled, element-free Galerkin, finite element approach. The concrete was modeled with particles, via a continuum damage model, where an anisotropic tensile damage variable was introduced to capture the behavior of concrete in tension. The reinforcement was discretized with beam elements, in which an elastoplastic constitutive law with isotropic hardening and a tension cutoff was applied. The relative displacement between the reinforcement and the adjacent concrete particles was calculated, and no interface elements were introduced. When compared with two experiments with different failure mechanisms, a bending failure and a shear failure, the two-dimensional model was validated. However, for structures without constant thickness, this model cannot reproduce the appropriate, complete failure mechanism.

Baxi (2005) presented an in-depth analytical study of the bond behavior of strands in the end zone of pre-tensioned concrete girders. An axi-symmetric FE analysis of concrete cylinders using ABAQUS was conducted to investigate the state of stress in the concrete surrounding the strands just after transfer of prestress. Both fully bonded and debonded models were considered. The modeling approach allowed for slip between the strand and the concrete within the development length of the prestressed concrete members. The FE analysis also allowed for modeling the cracking and crushing of concrete.

Vecchio et al. (2006) studied the behavior of post-tensioned concrete beams with FE analysis. The post-tensioning tendons were represented by truss bar elements and the concrete by solid elements. A stress-slip model was defined to describe the differential movement of the tendon elements relative to the surrounding concrete elements. It was shown that neglecting friction effects in unbonded post-tensioned beams had a minor influence on the computed response, whereas ignoring tendon force increases due to additional straining under loading may cause a significant under-estimation of load capacity in shear-critical beams.

Based on the FE program DIANA, Bolmsvik and Lundgren (2006) studied the bond mechanism between strands and concrete and how different detailing of the strand interface affected the behavior. A bond model was calibrated by use of pull-through tests. It was shown that the initial bond response was mainly attributed to adhesion in the strand-concrete interface. Regarding indented strands, the maximum bond capacity was determined by the strand indentation.

The fully-bonded FE model was established based on a recent research project at the University of Tennessee (Vadivelu 2009) to analyze the performance of a prestressed concrete I-girder as shown in **Figure 1.17**. In this model, the girder concrete was meshed with 20-noded quadratic brick elements and the strands were modeled with 3-node, quadratic 3-dimensional (3D) truss elements. The contact between the concrete and the strands were applied using a method called the “embedded element technique”. The embedded element technique was used to specify an element or a group of elements that lay embedded in a group of host elements whose response would be used to constrain the translational degree of freedom of the embedded nodes i.e., nodes of the embedded elements. All of the host elements can have only translational degrees of

freedom which must be identical to the number of translational degrees of freedom at a node on the host element.

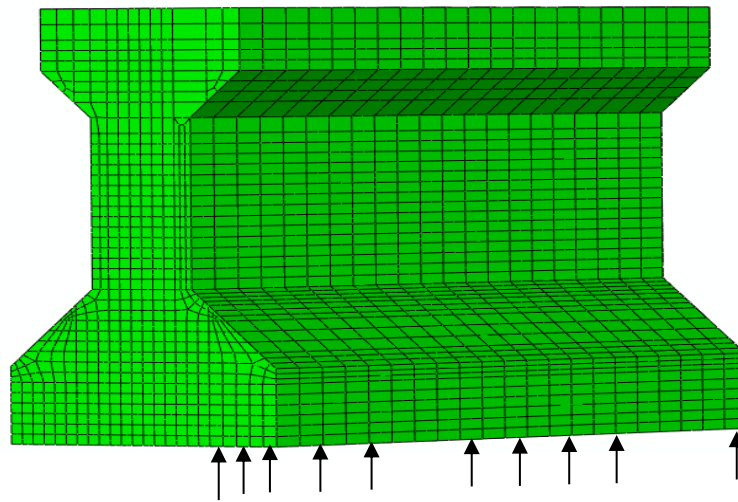


Figure 1.17 - Finite element model of a prestressed concrete I-girder (Vadivelu 2009)

Prestressing force was the only external force considered for this analysis, and was introduced by applying an initial compressive stress to the strand elements within the transfer length of the girder. The FE model was calibrated by using transfer-length data from two AASHTO Type I girders constructed with larger diameter strands. The calibrated FE model can be used to predict the distribution of stresses from the strands to concrete, and to predict where the concrete may crack. The example prepared by Vadivelu (2009), shown in **Figure 1.18**, predicts concentrated tensile stresses exist at the transition zone between the bottom flange and the web of an AASHTO type I-girder. This distribution revealed the cracking potential in the end zone of the girder which was confirmed in the testing.

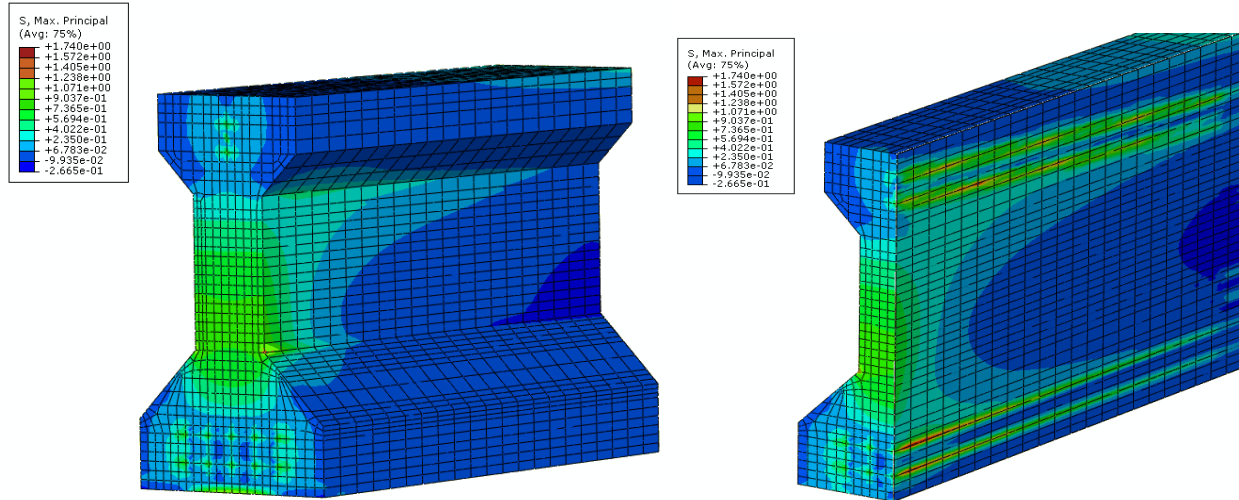


Figure 1.18 - Maximum principal stress distribution near the end zone with 0.7-inch strand (Vadivelu 2009)

Ayoub and Filippou (2010) introduced a nonlinear FE model for pretensioned prestressed concrete girders. The model consisted of three components: a fiber beam-column element for concrete, a truss element for prestressing tendons, and a bond element that described the transfer of stresses between tendons and surrounding concrete. This model was based on a two-field mixed formulation, where both forces and deformations were approximated within the element. The stress transfer mechanism was modeled with a distributed interface element with special bond stress-slip relation. The experimental results confirmed the accuracy and efficiency of the FE model.

Arab et al. (2011) presented a three-dimensional FE simulation of pre-tensioned concrete members, and its validity was verified by experimental results. Two approaches were examined for FE modeling: the extrusion technique utilizing friction-based contact simulations, and the embedment technique simulating equivalent responses. By comparison, the former technique provided more detailed information including the interface over stresses and bond slippage while the latter was a computationally less expensive alternate to simulate the overall response with comparable accuracy.

Burgueno and Sun (2011) analyzed the effects of debonded strands on the production and performance of prestressed concrete beams. The proposed nonlinear FE models, simulating the bond between concrete and strands, were calibrated by small-scale prestressed concrete beam tests. Three FE models of AASHTO box girders were analyzed to study an incident of end cracking in the manufacturing of a bridge girder. The effects of different debonding methods were also discussed through FE method with ABAQUS.

1.6 Crack Development in Reinforced Concrete Beams and Shear Transfer through the Cracked Section

1.6.1 Truss Angulation Models

The illustration in **Figure 1.19** by Mitchell and Collins (1991) presents the two early tests conducted by Morsch. These illustrations present the behavior of unreinforced and reinforced concrete beams under a distributed load after developing diagonal cracking. Each step in the illustration shows similar cracking patterns, but for the reinforced concrete member the cracking occurs at an elevated loading when compared to the unreinforced section. The unreinforced section failed with a 24-ton loading while the reinforced beam failed with a loading of 42 tons. The unreinforced beam encountered a sudden shear failure while the stirrups in the reinforced section transferred the shear to the stirrups and failed with a flexural failure at a much higher loading.

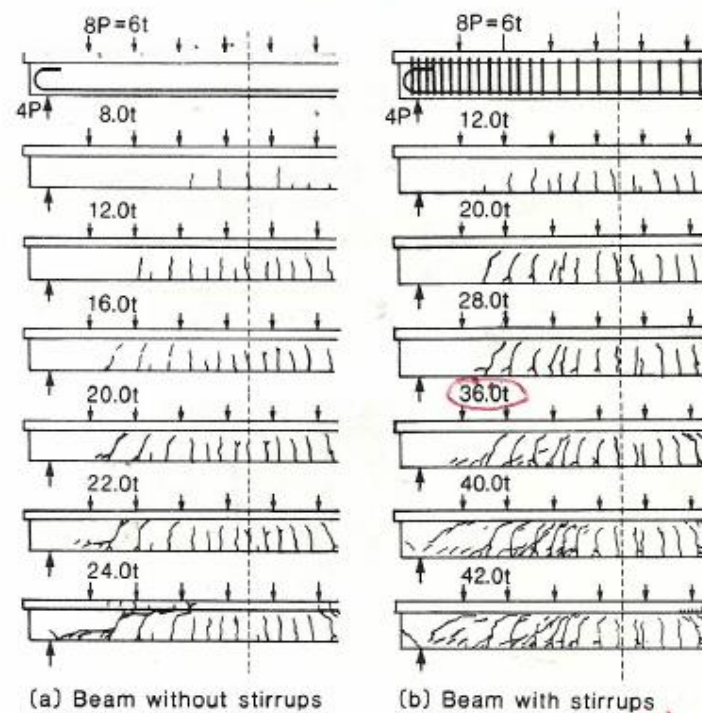


Figure 1.19 - Tests of Simply-Supported T-beams Subjected to Uniformly Distributed Loads. Adapted from Morsch and Collins and Mitchell (1991)

This testing formed the basis of shear design. The objective of shear design is to avoid premature brittle failures as displayed by the beams without stirrups. Members are to be designed with sufficient shear capacity to ensure a ductile failure as exhibited by the beams with stirrups. (Collins and Mitchell, 1991)

In the late 1960's and early 1970's, many researchers including Fenwick, Pauley, Gergeley, Leonhardt, Walther, and Taylor published work concerning the distribution of shear stress across the cross section of an unreinforced concrete beam. These findings were summarized in Taylor's (1970) literature review in his report. Through testing, Taylor published that his work was in agreement with other work on dowel and compression zone forces. Taylor and others contributed shear stress in unreinforced concrete members to three different sections of an unreinforced concrete beam. They were able to easily measure the average surface strain in the compression zone. They found the shear strain resisted in the longitudinal (dowels) reinforcement. The remainder was attributed to the center (web) section of the beam. Early researchers considered that this section, when cracked, would contribute very little to the shear resistance of a concrete girder and was often neglected. Through testing, Taylor published the following result:

Shear Capacity Percentage by Section (Taylor, 1970)

Compression Zone	20 – 40%
Aggregate Interlock	33 – 50%
Dowel Action	15 – 25%

This testing demonstrated that even though cracking existed, a significant amount of shear resistance was generated in the web section at $\approx 41.5\%$ of the capacity of the beam subsequent to cracking.

Figure 1.20, from Mitchell and Collins (1991), presents a model showing the diagonal truss field as it is developed with stirrups crossing the cracks developed under applied load. The initial assumption from the researchers was that the compression field could be described as a 45 degree angle. This would give the most conservative value of the tensile force within the stirrups. Consider the free body diagram presented in **Figure 1.20b**. The shear resistance in the web of the T-beam is modeled by visualizing angulated, concrete columns restrained by the vertical stirrups. Using this model, the shear in the section is distributed across the surface area of the concrete columns at the section of interest. In this case, the surface area can be described as $jd/\sqrt{2}$ or $d_v/\sqrt{2}$. Assuming the angle of inclination of the stress field is 45 degrees, the average stress in the concrete can be determined and the stress divided by the area included in the spacing of the vertical stirrups can be calculated. By equilibrium, the vertical component of the stress field will equal the resistance in the vertical stirrups which is $A_v f_v$ for a given spacing. This is illustrated in **Figure 1.20c**.

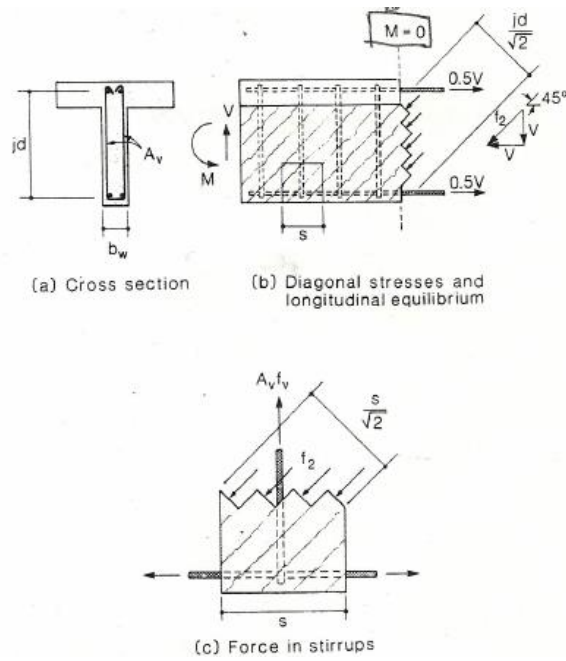


Figure 1.20 – Equilibrium Considerations for 45° Truss Model (Mitchell and Collins 1991)

According to Mitchell and Collins (1991), Richart conducted many shear tests at the University of Illinois. In this testing, Richart noted that the 45° for the direction of principal compression might be the cause of the reduced strain in the stirrups that were observed in the beam tests. Richart pointed out that reducing the angulation of the concrete compression field to 40° would reduce the predicted strain in the stirrups by 20%.

In **Figure 1.21**, Mitchell and Collins (1991) present the same model as **Figure 1.20** but considering a variable truss angle. With the reduced angle, less of the shear at the section is transferred to the vertical stirrups and more to the longitudinal reinforcement. From this general free body diagram, equilibrium equations can be written which correlate the conditions within the beam to the angle of the compression field to the longitudinal reinforcement. As an example, the stress in the concrete (f_2) can be given as such:

$$f_2 = \frac{V}{b_v d_v} \frac{1}{\sin \theta \cos \theta} \quad (1.3)$$

where the vertical component of the shear is $V/\sin \theta$ and the area is expressed as $d_v \cos \theta$.

Additionally, the longitudinal component of the diagonal compressive force due to shear is

$$N_v = V \cot \theta = A_{Longitudinal\ Steel} f_{Long\ Stl} + A_{Conc} f_{Conc} \quad (1.4)$$

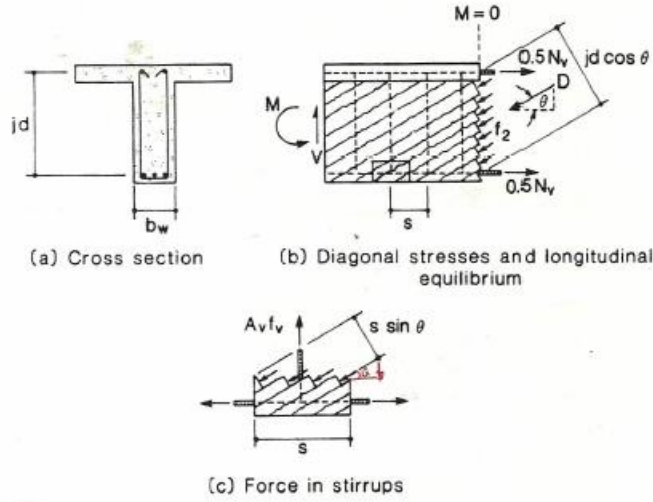


Figure 1.21 - Equilibrium Conditions for Variable-Angle Truss, Mitchell and Collins (1991)

A third equation is illustrated in **Figure 1.21c**. The outward thrust of the stress created by the stress in the concrete, created by the shear in the system, must be equilibrated by the stress in the stirrup. This gives the following:

$$f_2 b_v s \sin^2 \theta = A_v f_v \quad (1.5)$$

Substituting,

$$f_2 = \frac{V}{b_v d_v} \frac{1}{\sin \theta \cos \theta} \quad (1.6)$$

$$\frac{A_v f_v}{s} = \frac{V}{d_v \tan \theta} \quad (1.7)$$

These equilibrium equations relate four variables in three equations. These equations cannot be solved algebraically without introducing other intrinsic relationships to solve the variables.

In 1964, Kupfer developed a procedure for determining the angle θ using a minimum energy procedure. According to Mitchell and Collins (1991), this deviation used the linear elastic properties of the steel and concrete and realizing steel is much stiffer than the concrete, it was capable of carrying more of the shear stress, therefore θ had to be less than 45° .

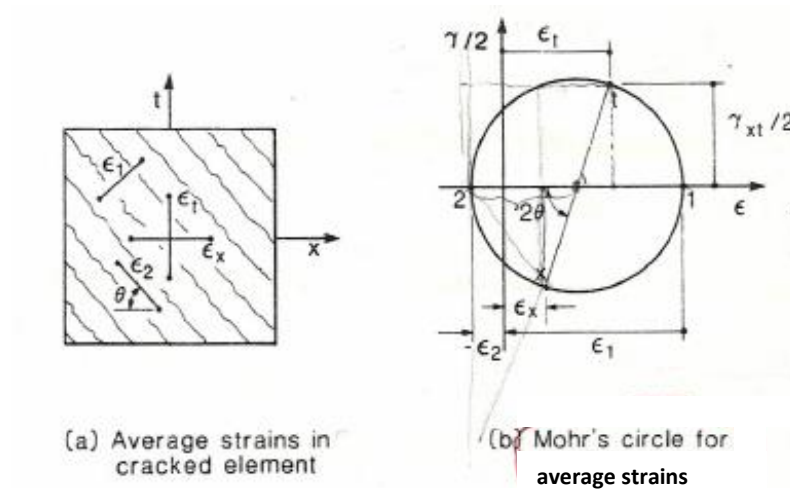
1.6.2 Compression Field Theory and the Contribution of Tensile Stress across Concrete Cracks

Another refinement of the truss model is the compression theory and the modified compression theory as developed by Collins, Mitchell, and Vecchio in the 1980's. Their attempt was to take the current truss models and using work conducted by Taylor, regarding mechanical aggregate interlock; Kupfer, regarding the minimum potential energy concepts to derive more accurate crack angle equations; and work that they completed to quantify the tensile stress component that crossed concrete cracks, developed their model for evaluating shear in a reinforced and prestressed concrete beam. They developed new material properties for cracked concrete, took into account the variability of the crack angulation, and resolved equilibrium, compatibility and constitutive relationships within a concrete specimen and formulated them in terms of average stresses and average strains. (Vecchio and Collins, 1988) Collins and Mitchell understood that this level of analysis, at the time, was "too complex for regular use in the design of simple beams," but that "the procedure has value in its ability to provide a rational method of analysis and design for members having unusual or complex geometry or loading, or whenever a more thorough analysis is warranted." Since the 1980's, Bentz and Collins have developed a computerized program called "RESPONSE" which is a design program based upon their research that simplified this effort. This modified compression theory method has been incorporated into the American Association of State Highway Officials (AASHTO) Load and Resistance Factor Design (LRFD) manual.

Because of the complexity of the modified compression theory, the ACI-American Association of Civil Engineers (ASCE) Shear Committee 426 desired to seek a rational simplification of the shear design procedures. Their desire was to have a simplified procedure that would provide an adequate level of safety, be correct in basic concept, although not necessarily accurate in details, be simple to understand and use, mathematically derived, and not necessarily increases design or construction costs. Based upon this intent, Ma and Tadros (1999) developed simplified empirical equations to quantify approximations of the effective shear depth (d_v), factor indicating the ability of diagonally-cracked concrete to transmit tension (β), and the angle of inclination of diagonal compressive stresses (θ). In their work, Ma and Tadros demonstrated that their empirical equations approximated the results obtained from modified compression theory, provided a safe and simple method for evaluating shear within a concrete beam, did not require charts and graphs to evaluate a beam, and was not an iterative procedure.

Before the equilibrium equations of the variable-angle truss model can be used to design a member for shear, the inclination angle θ must be known (Mitchell and Collins, 1991). Wagner, in studying postbuckling shear, assessed that the thin web would not resist compression and shear would be resisted by diagonal tension. The inclination angle of the diagonal tensile stresses would coincide with the inclination of principal tensile strains.

The compression field theory applies this approach to reinforced concrete and, upon cracking, the assumption is that the concrete carries no tension and the shear is carried by a field of diagonal compression. Using this assumption, Mohr's circle can be used to develop an expression for the shear that is carried by a field of diagonal compression. **Figure 1.22** presents Mohr's circle of strain compatibility using the average strain across several tension cracks.



**Figure 1.22 – Compatibility Conditions for Cracked Web Element
Mitchell and Collins (1991)**

Using Mohr's Circle we understand that

$$\cos 2\theta = \frac{\left[\frac{\epsilon_1 - \epsilon_2}{2} - \epsilon_x + \epsilon_2 \right]}{\left[\frac{\epsilon_1 - \epsilon_2}{2} \right]} \quad (1.6)$$

$$= [\epsilon_1 - \epsilon_2 - 2\epsilon_x + 2\epsilon_2] / [\epsilon_1 - \epsilon_2] \quad (1.7)$$

$$= [\epsilon_1 - 2\epsilon_x + \epsilon_2] / [\epsilon_1 - \epsilon_2] \quad (1.8)$$

$$\tan^2 \theta = (1 - \cos 2\theta) / (1 + \cos 2\theta) \quad (1.9)$$

$$= \{ [1 - (\epsilon_1 - 2\epsilon_x + \epsilon_2) / (\epsilon_1 - \epsilon_2)] / [1 + (\epsilon_1 - 2\epsilon_x + \epsilon_2) / (\epsilon_1 - \epsilon_2)] \} \quad (1.10)$$

$$= (\epsilon_1 - \epsilon_2 - \epsilon_1 + 2\epsilon_x - \epsilon_2) / (\epsilon_1 - \epsilon_2 + \epsilon_1 - 2\epsilon_x + \epsilon_2) \quad (1.11)$$

$$= (\epsilon_x - \epsilon_2) / (\epsilon_1 - \epsilon_x) \quad (1.12)$$

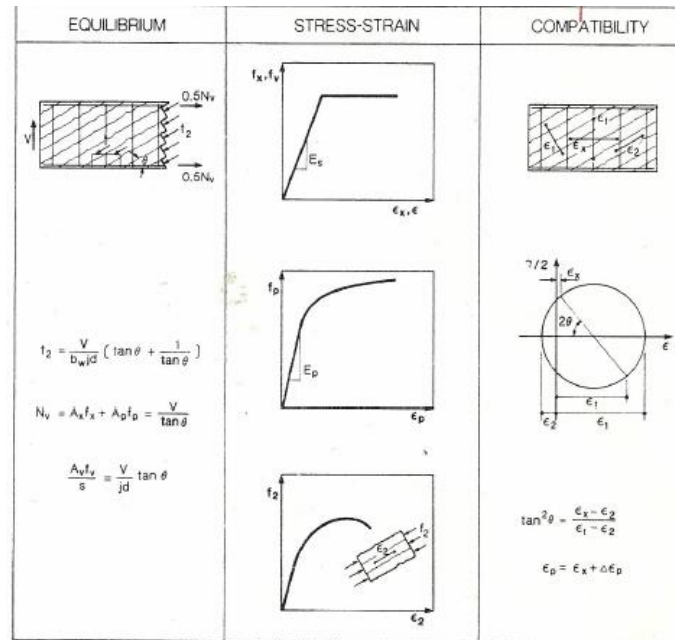
and,

$$\epsilon_1 = \epsilon_x + \epsilon_t - \epsilon_2 \quad (1.13)$$

so,

$$\tan^2 \theta = (\epsilon_x - \epsilon_2) / (\epsilon_1 - \epsilon_2) \quad (1.14)$$

With compatibility, two more equations and one more unknown were added. Knowing this and coupled with the relationships between stress and strain, a solution can be attained algebraically. This approach is known as the Compression Field Theory. **Figure 1.23** summarizes the equilibrium, stress-strain, and compatibility relationships for the compression field theory.



**Figure 1.23 – Summary of Property Relationships in Reinforced Concrete Beam
Mitchell and Collins (1991)**

1.6.3 Predicting Response in Shear using Modified Compression Theory

The primary difference between the Compression Theory and the Modified Compression Theory is that the Compression Theory neglects the contribution of tensile stresses in cracked concrete and a contribution is included in the modified compression theory. Prior to cracking, shear is carried by both diagonal tensile and compressive stress. After diagonal cracking, the tensile stress in the concrete is substantially reduced and transferred across the cracks. The tensile stresses in the concrete vary from zero at the cracks to a maximum at the midpoint between the cracks.

The diagonal compressive stresses push apart the flanges of the beam while the tensile stresses pull them together. Based upon testing conducted by Vecchio and Collins, equations for the average strain across the cracks have been developed which relate the average strains in these zones, the bond characteristics of the reinforcement and the type of loading.

1.7 Summary of Larger Diameter Strand Research

Vadivelu (2009), in his Master's thesis presented a case for the use of larger diameter strand. In this thesis, he presented the results of a parametric study using 0.7-inch 270 ksi strand and concluded that when using this strand at a 2-inch spacing a considerable savings in flexural member material could be realized. To determine the adequacy of the strand, Vadivelu developed a 3-dimensional finite element model examining the effects that the additional prestressing force had on AASHTO Type I girders. Principal and axial stresses were examined and based upon the modeling; it was found that the 0.7-inch strand beams with 2-inch spacing would likely be more vulnerable to cracking at the transition zone between the bottom flange and web. The study recommended the use of confining steel to overcome this tendency. Following this study, two I-girder specimens were cast, one with 0.7-inch 270 ksi strand and the other with 0.62-inch 330 ksi strand. The transfer lengths were measured and it was found that the transfer lengths for both specimens were shorter than the lengths estimated using AASHTO 2008 and ACI 318-08 equations.

Akhnoukh (2008) looked at the use of 0.7-inch 270 ksi strand. The specific items addressed were as follows: 1) Determine the required confinement steel necessary that would allow the determination of transfer and development lengths according to current procedures in the AASHTO LRFD Bridge Design Specifications for smaller strands; 2) Develop a 15 ksi self-consolidating mix design with a one day compressive strength of 10 ksi; and 3) Test the use of 80 ksi welded wire reinforcement as auxiliary reinforcement for shear, web end splitting, and confinement.

Akhnoukh used shear friction theory to estimate the level of confinement required to comply with the AASHTO LRFD specification and developed a simplified mathematical expression to calculate the area of confining steel required as a function of prestressing, concrete strength, and strand distribution. Akhnoukh observed that the transfer length of 0.7-inch strand was shorter than the lengths predicted by AASHTO LRFD specifications. He noted that the level of confinement had only a slight impact on transfer length. A successful mix design was developed and he had success using 80 ksi, welded wire reinforcement in his design.

Another parametric study was conducted by Ferhadi and Badie (2010) looking at extending span lengths of prestressed I-girders using high performance concrete, high-strength light-weight concrete, large-diameter, high-strength strand, and bridges made continuous for slab weight and superimposed loads. They recommended the use of 0.7-inch strands with normal-weight concrete, rather than light-weight concrete, which would offer better performance for the difference in cost. For 0.6-inch strand, using light-weight concrete is more economical.

Patzlaff (2010) conducted a study on the impact of bottom flange confinement on the performance of prestressed concrete bridge girders. Transfer length, development length, and vertical shear capacity were examined using eight tee-girders and three NU1100 girders. The

results of the study indicated that the amount or distribution of confinement reinforcement did not have a significant effect on the initial or final transfer length of the prestressing strands. Additionally, no significant impact of confinement was found on either the nominal flexural capacity or bond capacity of the prestressing steel, and the effect of confinement on the shear resistance of the girder was negligible. Confinement across the entire girder was observed to increase the ductility of the girder and reduced cracking under extreme loading conditions.

Hatami, Morcou, and Tadros (2011) evaluated the bond performance of 0.7-inch diameter, 270 ksi, low-relaxation strands in mortar and concrete using the North American Strand Producer (NASP) bond test method. Fifty-eight samples were tested and an equation that predicted the bond in mortar and concrete was developed. Tests were conducted on both clean and rusted strands. They found that the NASP test method can be applied to 0.7-inch strand, the performance of rusted strand was 40% higher than clean strand except during end slip conditions. At end slip conditions, they found that the performance of rusted strands could be highly variable and performance lower than clean strands.

Tadros, and Morcou (2011) conducted an examination of the use of 0.7-inch strands for their use in bridge girders. Several experimental investigations were conducted to determine the mechanical properties of 0.7-inch diameter strands, address production concerns, evaluate transfer and development length, and end zone cracking associated with using these larger diameter strands. Specimens included rectangular prisms, 24-inch deep T-beams, NU900 and NU1100 girders. The work indicated that girders could be produced using 0.7-inch diameter strands with no major changes to current production practices or to the current design criteria according to AASHTO LRFD specifications.

Morcou, Hanna, and Tadros (2011) report that 0.7-inch strand is stiffer and heavier, and producers should be aware of this when using this strand. Some modifications will have to be made to enlarge bulkheads and increase the jacking capacity of the equipment or prestressing bed. They noticed a reduction of the predicted transfer length to $50d_b$ instead of the $60d_b$ predicted by using AASHTO LRFD specifications. Further, they defined values of strand spacing, concrete strength, and confinement to develop the 0.7-inch strand.

1.8 Recent Shear Design Approaches in Prestressed Concrete Members

Ma, Tadros, and Baishya (2000) conducted a study into the shear behavior of pretensioned high-strength concrete bridge I-girders. Current AASHTO Standard specifications require that V_s not exceed $8\sqrt{f'_c} b_w d_v$ (f'_c in psi) without otherwise widening the section web or increasing the depth of web. Also, a maximum shear limit of $0.25 f'_c b_w d_v$ is required by AASHTO LRFD Specifications. These two limits were examined and it was observed that as high-strength concrete was used, the V_s limitation would unduly widen the prestressed concrete member. An experimental testing program was created to examine these limits using full-scale beams and high-strength concrete. Strand draping and shielding were also used as parameter in the testing program.

It was determined that both limits provided conservative results when compared with the testing data. The tests produced shear capacities in excess of the AASHTO LRFD limits with shear reinforcement less than the maximum allowed. The AASHTO-STD maximum allowable shear limit appears to be too restrictive, which may result in unnecessary widening of the beam web. Anchoring strands within end blocks or end diaphragms can significantly improve the shear resistance of a beam. Beams without end anchorage have significantly lower shear capacity. The test results confirmed that the $0.25 f'_c b_w d_v$ value is a reasonable limit for concrete shear resistance.

Ma and Tadros (1999) developed a simplified (non-iterative) approach to shear design. This was a goal set by the ACI-ASCE Shear Committee 426 for the simplification of concrete shear design. The procedure applies to members not subjected to axial tension and having the minimal amount of transverse reinforcement requirements specified in Article 5.8.2.5 of the AASHTO LRFD specifications. The β_1 term is simplified as the multiplication of γ_a and γ_p . γ_a is dependent on the beam's M_u/M_{cr} ratio and γ_p is dependent on the ratio of prestressing stress to the concrete compressive strength. The angle of inclination of diagonal compressive stresses is also given in terms of γ_a and γ_p . The simplified procedure was validated using testing results from previous shear testing programs and matched the results given by the iterative calculation procedure from AASHTO LRFD.

Kuchma, et al. (2008) conducted shear testing on several high-strength prestressed bulb-tee sections and compared the measured shear strength capacities to five different methods of shear capacity determination. The methods compared were the sectional design model of AASHTO LRFD, the Canadian Standards Association design of Concrete Structures, Response Program 2000 (a sectional analysis program), AASHTO Standard Specifications, 17th edition, and simplified design provisions of the LRFD specifications. The report concluded that the compressive strength limit of concrete can be increased from 10 ksi to 18 ksi without impacting safety. They recommended the use of strut-and-tie modeling be used in lieu of a sectional approach when the shear design stresses exceeded $0.18 f'_c$ unless the member is cast integrally with the support or is continuous over the support. They observed that the calculated shear capacity of the test girders was conservative for the AASHTO LRFD, AASHTO STD, and Response Program 2000. They reported that AASHTO LRFD approximated the angle of inclination and initial position for the shear cracking appropriately. They observed that significant cracking and shear reinforcement yielding occurred when the beam was designed to support shear stresses in excess of $0.15 f'_c$.

Concrete beam failure can occur in a number of ways. Ma and Tadros (1999) summarized the failure mechanisms as follows:

1. The compression zone is reduced by the increasing vertical flexural crack and finally is crushed in compression (a typical flexural type failure).
2. A diagonal crack extends at both ends towards the flanges, and finally the beam splits into two pieces (a diagonal tension failure).
3. The compression zone is reduced and crushed in compression by the extended diagonal crack (a diagonal compression failure).

4. Both diagonal and vertical cracks widen, but spalling appears along the longitudinal reinforcement, and finally the beam loses its capacity (a typical bond failure).
5. Diagonal cracks appear in the web of the beam near the support, spacing between the almost parallel diagonal crack is reduced, spalling of the web surface appears, and the web concrete is crushed (a typical web-crushing failure).

In addition to these, if appropriate development length of the longitudinal reinforcement is not attained before the necessary tensile capacity demand is met, the failure can also occur due to insufficient end anchorage of this reinforcement.

2.0 MODEL FOR STRAND BEHAVIOR IN PRESTRESSED CONCRETE

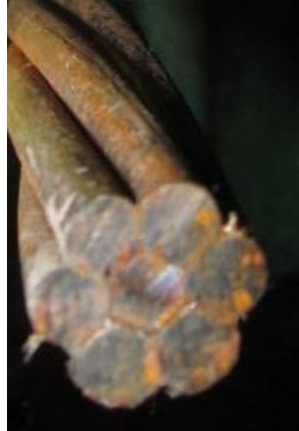
The behavior of strand within concrete is a complex series of time-dependent processes which is influenced by the material properties of concrete and strand. The best way to understand the process is to separate the interactions that occur and understand each individual component. This chapter examines each component and then combines them into a mathematic model to describe strand behavior.

2.1 Strand Properties

Strand is a seven-wire, stiff steel component that is manufactured using rod stock. Steel wire stock is heated, pulled, reduced in size and wrapped in a helical fashion while the strand is still hot. Though not welded together, the individual strands are pulled while being wound around the center strand, and the contact surfaces are pressed together to form a slight bond. Strand with a nominal 0.70-inch diameter is typically made from 0.26-inch diameter wires. This combination results in approximately 0.08 inches of material being compressed together across the diameter. Though this bond is substantial, it has been observed, under ultimate loading, that the center wire can slip relative to the other six wires. Strand, after being spun together, is reheated and pulled. The pulling tension applies residual stress to the strand allowing it to unroll straight when unspooled.

The outer six wires are spun in a helical pattern. The wires were measured to depart from the longitudinal axis of the strand by approximately 14 degrees. When the strand is cut perpendicularly to the longitudinal axis, a cross section is created as shown in **Figure 2.1**. The cross section of the exterior strands is slightly elliptical due to the helical pattern. The cross sectional area for 0.70-inch strand is 0.2923 in² as reported by Insteel Wire Products. (See **Appendix A** for a copy of the Material Certifications for the 0.62- and 0.7-inch strand used in this investigation.) The contact surface area was estimated graphically with AutoCAD and was found to be 2.911 in²/in. Also, the contact surface area for 0.5- and 0.6-inch strand was estimated to be 2.083 in²/in and 2.497 in²/in respectively using the same graphical technique.

The material certification reports also report a load for a 1% extension of the strand as well as the ultimate elongation cross sectional area and observed modulus of elasticity. The steel delivered is listed as 7W 270 LR ASTM A416. The Poisson's ratio used was 0.3 which was typically used in publications that were examined.



**Figure 2.1 – Photograph of
0.7-inch Strand**

Strand behaves similarly to steel rod when being elongated. As longitudinal stress is placed upon the strand, the strand elongates and shrinks longitudinally. This variance is proportional in the elastic range of steel and occurs at a uniform rate. This rate, Poisson's Ratio, can be described mathematically as follows:

$$\nu = - d\epsilon_{\text{transverse}} / d\epsilon_{\text{axial}} \quad (2.1)$$

where,

ν is the resulting ratio,
 $\epsilon_{\text{transverse}}$ is the transverse strain,
 ϵ_{axial} is the axial strain.

Once the strand is elongated—prior to casting within a concrete member—the strand shrinks radially. It is customary to prestress the strand by elongating the strand to a stress of 75% of its ultimate stress (AASHTO specification). Though some of this prestress is lost in the manufacturing process, through concrete creep, steel relaxation, concrete shrinkage, and thermal differences, a majority of the stress is retained in the strand. These losses typically account for a stress reduction of approximately 20 to 25% of the original stress level. The remaining stress level then becomes 56 to 60% of the yield stress of the strand.

After the concrete is cast and allowed to set to the designated release strength, the strand is cut thus releasing the tensile energy applied to the strand. When this occurs, the stress in the strand at the end of the section returns to zero and the stress within the strand is transferred to the section through bonding and frictional forces within the cast concrete. The stress is variable from the end of the section to the transfer point. The transfer point is defined as the position where the initial stress which was applied to the strand—less prestress losses described earlier—is transferred to the concrete.

2.2 Concrete Properties

Concrete, in its simplest form, is a mixture of aggregate—both large and small—and a binding agent. Portland cement paste is generally the most common binder used in the manufacture of concrete. It mainly consists of oxides of calcium, silicon and aluminum with a source of sulfate. These components are heated and ground to formulate dried cement.

When water is added, a chemical reaction ensues. This reaction (hydration) glues the aggregate and reinforcement together. Many chemical reactions take place over time including the formation of Calcium-Silica-Hydrates (C-S-H) and Ettringite. These molecules shown in **Figure 2.2** develop long finger-like crystalline appendages which attach themselves to any embedded material.

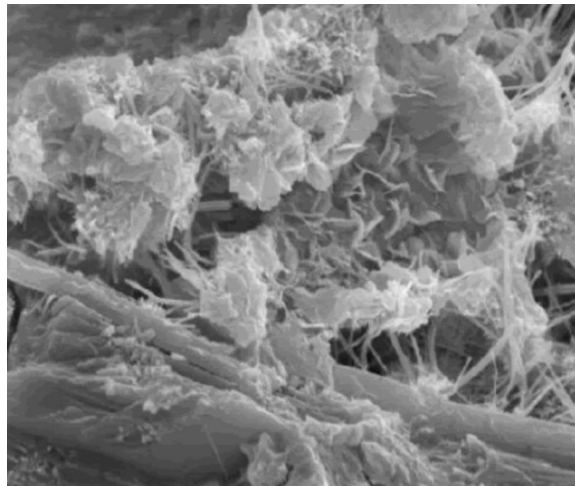


Figure 2.2 – C-S-H and Ettringite Molecules from FWA Website

Consider **Figure 2.2**. For the sake of visualization, the cement molecules can bind themselves to strand in the form of long slender microscopic columns. Several correlations are apparent in the macroscopic world. For instance, the column density along the length of the strand is akin to the concrete strength. If the cement columns were compressed, the adjacent columns would act as lateral bracing against themselves. If the cement columns were pulled, lateral bracing from the adjoining columns would not occur and the column would have to rely only upon its slender cross-sectional area to resist the tensile forces. Precompressing the concrete would be akin to increasing the density and adding lateral bracing between the microscopic cement columns. Shear behavior can be described similarly. With an increase in cement column density an increase in the shear strength of the concrete can be realized. If the columns were confined with lateral pressure, then the columns would not collapse as easily and the relative strength of the system would be increased.

This is a visualization tool, but it is useful in describing the development of a concrete system behavior. It is based upon the microscopic properties of concrete and can be used to develop a model for concrete behavior.

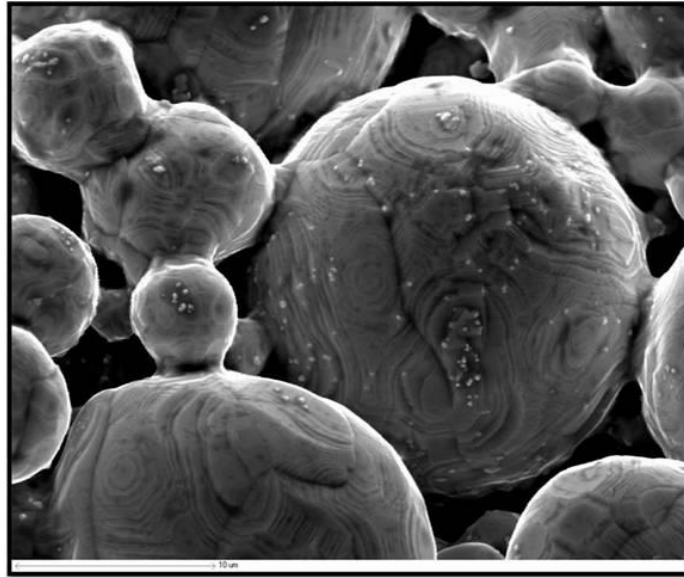


Figure 2.3 – Microscopic Picture of Steel Molecules from Artword.net

Because of the molecular shape of concrete, it behaves much differently than steel which has a more compacted and round microstructure—see **Figure 2.3**. Steel particles are welded together to form their bond and are tightly packed confining each molecule together. As a result, steel is much stronger than concrete particularly in tension. Also, if steel components are shaped as long slender elements, then steel would behave similar to long slender concrete.

Included in **Appendix B** is a copy of the concrete batch reports and aggregate tests for the concrete used in the small-scale testing.

2.3 Interaction between Strand and Concrete

In literature, three types of strand and concrete interaction have been identified. These three are adhesion, mechanical interlock and a “Hoyer” effect. Adhesion describes how well the cement paste sticks to the embedded strand. Mechanical interlock describes how deformities in the strand interfere with the concrete when being pulled through concrete. The Hoyer effect describes the normal and frictional forces being developed between the concrete and strand interface. As the strand is released from a casting bed, the strand will try to swell and shorten with the release of stress within the strand.

These elements provide resistance to strand movement within the cast concrete. Based upon testing conducted by Jiang, Ma, and Cabage (2013), the mechanical interlock resistance is less prevalent in strand than it is in deformed bar. Under extreme loading conditions, the strand was observed under many slip conditions to rotate through the concrete casting rather than the strand deformations interlocking with the concrete aggregate and providing additional resistance to strand movement.

When strand is stretched within a casting bed, it shrinks radially, typically as predicted by Poisson's ratio. When concrete is cast and allowed to cure, a casting is formed surrounding the strand. Once the strand is released, the strand will try to shorten and expand radially. The stress within the strand at the edge of the beam returns to zero and has been found to increase to its pretensioning stress at a sufficient embedded distance.

If it is assumed that only two of the three bond mechanisms are prevalent within strand, and if these mechanisms can be described numerically, then a mathematical model can be developed to describe the behavioral mechanism of strand.

For adhesion, several variables can be used to describe this bond mechanism. Among these are the surface contact area of the strand and concrete, the embedment length of the strand, and the strength of the concrete adhering to the surface of the strand. The resistance provided by adhesion could be described as proportional to each of these parameters. In other words, a larger diameter strand would have a larger contact surface area per unit length and the adhesion resistance would be expected to be larger. A section of strand embedded further within a beam would be expected to have a larger total adhesion resistance than one near the edge of the beam. Also, a concrete with higher strength would be more likely to have a higher adhesion resistance than a lower strength concrete.

Mathematically, the expression to describe the adhesive resistance over a unit length could take the following form:

$$A_{resistance} \propto (SA_{strand}) (L_{embed}) (f'_c) \quad (2.2)$$

Where,

$A_{resistance}$ is the adhesive resistance
 SA is the strand surface area of the strand per unit length, and
 f'_c is the compressive strength of the concrete.

At the same time, additional tensile resistance force is developed as the strand tries to expand against the surrounding concrete, creating a static frictional force at the strand/concrete interface. For simplicity, assuming that this behavior occurs in the elastic region for both concrete and steel at transfer, then the change in diameter of the steel can be described in terms of Poisson's ratio.

For a given longitudinal strain in the strand (ε_{strand}), a given radius of strand (r_{strand}), and a strand modulus of elasticity, the total radial deformation of the strand can be described in the following form:

$$\Delta r_{strand} = \Delta \varepsilon_{strand} (v) (r_{strand}) \quad (2.3)$$

This change in radius size is resisted by the surrounding concrete when the strand is released. The deformation in the concrete, assuming linear-elastic behavior, can be defined as the simple expression of deformation as follows:

$$\Delta_{concrete} = \frac{\Delta P L_c}{A_c E_c} \quad (2.4)$$

Where,

$\Delta_{concrete}$ = deformation in the concrete

P = force applied to the concrete

L_c = effective compressive length of concrete material which can be deformed

A_c = area to which the force is applied

E_c = modulus of elasticity of the concrete

And,

$$\Delta r_{strand} = \frac{\Delta P L_s}{A_s E_s} \quad (2.5)$$

Where,

Δr_{strand} = radial deflection of the steel

P = force applied to the steel

L_s = effective compressive length of radial steel which can be deflected

A_s = area to which the force is applied

E_s = modulus of elasticity of the steel

As a condition of compatibility, the force causing the deformation in the concrete and the force in the steel must be equal. With this pressure applied to the concrete, a static friction force is created which adds to the adhesion resistance. The total of these two components of bond determine the amount of longitudinal strain which can be transferred to the concrete over a given length.

In a first attempt to quantify the components of strand bonding, Jiang, Ma, and Cabage in 2013 conducted a series of tests to isolate the contribution of each process to stress transfer from prestressing strand to surrounding concrete. The following sections describe the test procedures

and the results of these findings. This work was conducted cooperatively with the majority of the work being conducted by Jiang (2013).

2.3.1 Specimen Fabrication and Test Setup

Single stranded concrete specimens were cast in reusable steel forms, along with the concrete cylinders for the concrete compressive strength test (see **Figure 2.4**) for this series of tests. The concrete was vibrated in place to reduce the potential of poorly-bonded concrete with a low water to cement ratio. The prisms were prepared and stored in the structural laboratory where the temperature was maintained at 76 °F. They were covered with two layers: wet burlap and plastic sheeting. The prisms were stripped after one day, and they were tested on the third day. In this way, the age of concrete at testing was two days. From preliminary concrete mix design testing, a concrete compressive strength of approximately 10,000 psi was predicted.

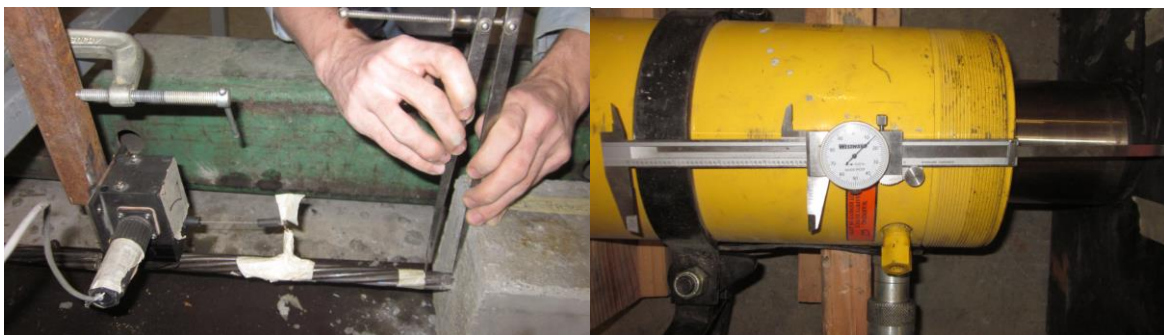
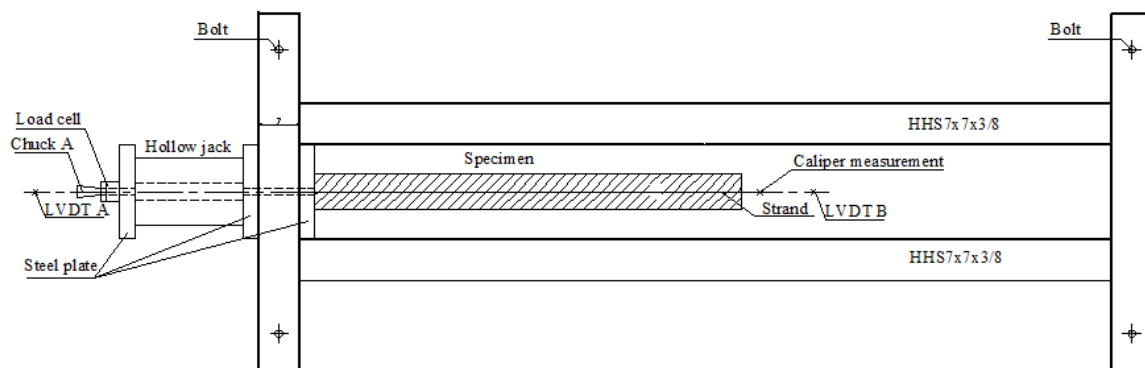


Figure 2.4 - Test Setup and Free-End Slip Measurement with String Potentiometer and Caliper

Two types of specimen sets at three different lengths were prepared. One set used non-pretension strand while the second used pretensioned strand. The purpose of this was to isolate the contribution caused by adhesion and radial expansion. The idea was to conduct pull-out tests using the same specimen size, concrete strength, and strand size and then compare the results of

each testing set. The difference between the two sets at the same length would be used to quantify the contribution of each component. 0.70-inch strand was used in this testing.

A DETachable MEchanical (DEMEC) strain measurement system was used to measure the concrete surface strain. The DEMEC strain points are small metallic discs of 0.25 inches in diameter, which were placed along the centerline of the prestressing strands and attached to two concrete surfaces. The gauge length of the points was 7.874 inches. Thus, the average concrete surface strain within the gauge length could be calculated through the change of the distance between two DEMEC strain points. The location of these strain points is shown in **Figure 2.5**. For the 12-inch specimen, two DEMEC strain points were symmetrically attached to the concrete surface along the specimen, and the spacing was the gauge length (7.874 inches). Thus, only one average strain in the middle length can be obtained for one side of the prism. For the 24-inch and 36-inch specimen, the spacing of two adjacent points is half of the gauge length (3.937 inches). The DEMEC gauges were symmetrically placed along the specimen length. Thus, four strain measurements were obtained for the 24-inch specimen on each side and seven strain measurements for the 36-inch specimen on each side. In the pull-out test, there were two ends: jacking end and free end. Strain 1 was the measured strain close to the jacking end while Strain 7 was at the free end.

The detailed information of the test setup is shown in **Figure 2.5**. As shown in this figure, a square-shaped horizontal steel frame was constructed using Hollow Structural Sections (HSS 7 x 7 x 3/8 inches). These sections were anchored to a strong floor with 1-1/4-inch-diameter threaded rods. A hollow-core hydraulic jack cylinder operated by a manual-controlled electrical pump was used to pull the strand, which was run concentrically through the steel frame and the hollow jack. A hollow load cell was placed behind the jack, and a chuck was against the load cell. Thus, a pull-out force was created with the elongation of the jack cylinder.

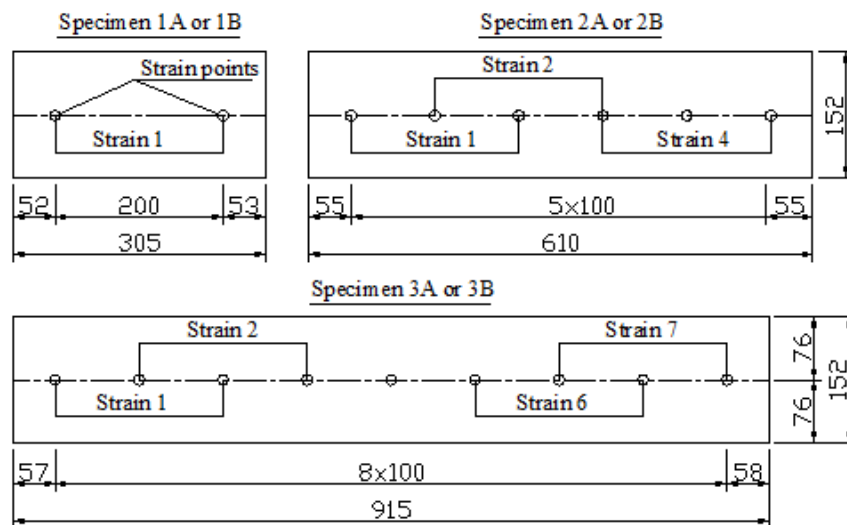


Figure 2.5 - The Location of the DEMEC Gauge Points

Measurements of pull-out force, end slip, and strain distribution were taken during the test. The force applied to the specimen was recorded from the load cell. The hydraulic jack was manually controlled and an incremental force was applied to the specimens. This pull-out force was increased by 5 to 10 kips for each step. Strand slip was measured at both the jacking-end slip and the free-end slip. String potentiometers were used to record the end slip. A metal hook attached to the strand with tape was connected to the string potentiometer. A micrometer was also used to measure the jack cylinder elongation to validate the electronic measurements. Strand draw-in was measured by placing a tape mark at the free end and using a caliper to determine the distance between the tape mark upon the strand and the end of the concrete specimen. This measurement was taken at each stage loading point and the difference between two measurements was the amount of draw-in for that staged loading. A string potentiometer was also used to determine this measurement electronically. However, when the pull-out force and the free-end slip increased, the free end of the strand would rotate, causing difficulty in attaining accurate measurements with the string potentiometer. The draw-in of the tape mark was therefore used to determine the free-end slip in lieu of the string potentiometer.

For the prestressed strand series of tests, the same testing apparatus was used as described previously except that the prestressing strand was initially pretensioned, the concrete cast and the strand released by saw-cutting the strand. The same concrete mix design used for the non-prestressed strand was used for the prestressed strand testing series and a 9,000 psi compressive strength was reached.

Three pretension levels were considered: 20, 40 and 60 kips. The specimens were cast and stored in the same environment as the non-pretensioned specimens. The prisms were stripped and the pretensioned strand was cut with a portable band saw within 24 hours of casting. The pullout tests were conducted within two days after the concrete was cast. In this way, the age of concrete on the pull-out test day was two days, which was the same for the non-pretensioned strand pull-out test. Therefore, the only difference of the pretensioned specimens and the non-pretensioned specimens was the development of the Hoyer effect at the free end and within the transfer zone. Through the comparison of the pull-out test for both non-pretensioned and pretensioned specimens, the bond mechanism was quantitatively investigated. In addition, with different length specimens and pretension force, the relationship between the transfer length and the pretension force level was studied.

Table 2.1 - Summary of Pull-out Test Results

Specimen ID	Length (in)	Pretension force (k)	Effective pretension force (k)	Draw-in before Pull-out (in)	Pull-out Force Activating Free-End Strand Slip(k)	Maximum Pull-out Force (k)	Maximum Free-End Slip (in)	Measured Maximum Concrete Surface Strain ($\mu\epsilon$)	Force Drop	Strand Rotation	Failure Mode
1-00A	12	0	0	0	1	34	0.260	154	Yes	No	Concrete split
1-00B	12	0	0	0	2	34	0.142	142	Yes	No	Concrete split
1-20	12	20	14	0.075	19	41	1.980	219	Yes	Yes	Strand slip
1-40	12	40	7	0.091	18	42	1.957	244	Yes	Yes	Strand slip
1-60	12	60	14	0.118	18	43	3.929	218	Yes	Yes	Strand slip
2-00A	24	0	0	0	7	59	0.961	290	Yes	Yes	Strand slip
2-00B	24	0	0	0	6	59	1.039	228	Yes	Yes	Strand slip
2-20	24	20	16	0.020	25	72	3.528	330	Yes	No	Strand slip
2-40	24	40	36	0.035	50	74	0.854	455	Yes	No	Strand slip
2-60	24	60	53	0.075	60	74	0.563	465	Yes	No	Strand slip
3-00A	36	0	0	0	13	76	0.063	274	Yes	No	Strand break
3-00B	36	0	0	0	14	79	0.130	302	Yes	No	Strand break
3-20	36	20	16	0.047	22	73	0.094	467	No	No	Strand break
3-40	36	40	29	0.063	45	83	0.098	442	No	No	Strand break
3-60	36	60	49	0.087	54	84	0.059	483	No	No	Strand break

2.3.2 Discussion of Testing Results

Table 2.1 presents a summary of the pull-out tests conducted primarily by Jiang (2013).

2.3.2.1 Discussion of the 12-inch Specimens

In the 1-foot-long specimens without pretensioning (1-00A and 1-00B), the ultimate pull-out force for these specimens was very close and the average was 34k; the free-end slip was 0.260 and 0.142 inches, respectively. The free-end slip began when the pull-out force was about 2 kips. Both the pull-out force and the slip gradually increased until the failure of the specimens. The relationship between the force and the free-end slip was parabolic while the jacking-end slip was approximately linearly and increased with loading. The concrete surface strain was distributed linearly with increasing load. The average concrete surface strain was approximately 150 microstrain when the load reached 34 kips.

Strand, when used for longitudinal reinforcement with a prestressed beam, is generally pretensioned to 75% of the strand ultimate strength. For the 0.70-inch-diameter strand, 75% of the strand ultimate strength was 60 kips. Three different pretensioning levels were used to investigate the effect of this force on the maximum pull-out force for the specimens. These different prestressing forces were 33%, 67% and 100% of the 75% of the ultimate strand strength limit. For the specimens 1-20, 1-40 and 1-60, the pretension force was 20, 40, 60 kips, respectively; the effective pretension force after strand release was 14, 7 and 14 kips.

Force drop during the pull-out process was detected when the pull-out force reached a certain level. The force drop extent for specimen 1-60 was more apparent than for specimens 1-40 and 1-20. For all pretensioned 12-inch specimens, the strand in the free end began to rotate when the pull-out force was about 38 kips. However, both force drop and strand rotation did not occur for the non-pretensioned specimens 1-00A and 1-00B. For all pretensioned 12-inch specimens, the strand in the free end did not slip at the beginning of the pull-out force. Instead, the free-end strand slip started when the pull-out force was increased from the initial effective strand force to 18k which was close to the pretension level for specimen 1-20. For the non-pretensioned specimens, the free-end slip occurred immediately following the application of pull-out force to the strand (see **Figure 2.6**). The pull-out force caused immediate damage to the chemical adhesion along the specimen; the adhesion appeared to contribute little to the bond. For the pretensioned specimens, assuming that chemical adhesion damage had already occurred, the resistance to pull-out force was attributed to the Hoyer's effect.

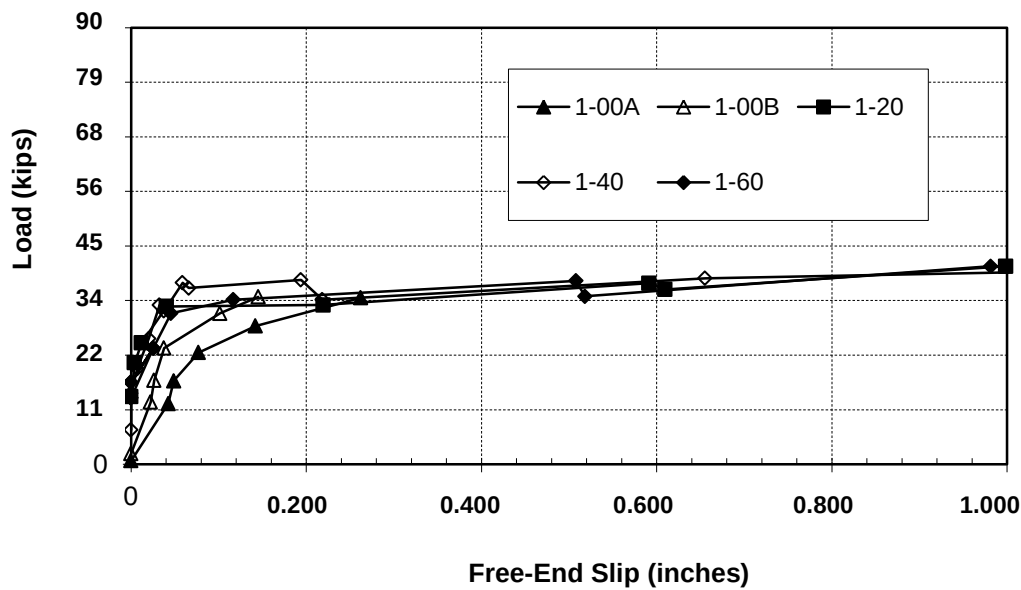


Figure 2.6 - Comparison of the Force-Slip Curves during the Pull-out Stage (12-inch Specimens)

2.3.2.2 Discussion of the 24-inch Specimens

Figure 2.7 shows the behavior of the five, 24-inch specimens when subjected to a pull-out force loading. In the non-pretensioned strand specimens (2-00A and 2-00B), the free-end slip occurred when the pull-out force was approximately 7 kips. This indicates that the adhesion force between the strand and the concrete was exceeded when the pull-out force reached 7 kips and the remaining resistance was due to a static frictional force between the concrete and strand. The maximum pull-out force for these specimens was 59 kips and the specimens could maintain a sustained load of 51 kips without slip. Once the sustained load was exceeded, free slippage occurred. It was apparent that the curve of the load vs. free-end slip was composed of two stages: the free-end slip slowly increased linearly before to a load of 45 kips with a slip of approximately 0.079 inches, then the curve reached a plateau where the slip drastically increased and the slope of the curve was much less than that in the previous stage. In the second stage, force drop occurred. For specimen 2-00A, the recorded first pull-out force drop occurred at 48 kips with a corresponding free-end slip of 0.283 inches; for specimen 2-00B, the recorded first pull-out force occurred at 52 kips with a free-end slip of 0.335 inches. When the force drop occurred, the loading was paused and held constant. The concrete surface strain measurements were then taken. Additional pull-out loading was then gradually applied to the strand, but a force drop did not occur.

During the stage of force drop, the free-end slip and the jacking-end slip increased at the same rate, indicating that the slip behavior occurred along the entire specimen. The free end strand

started to rotate when the pull-out force approximately 45 kips. As force was increased, the strand continued to rotate. The ultimate free end rotation for specimens 2-00A and 2-00B was about 9.5 and 7 degrees, respectively. The deviation between the strand centerline and the shadow of the string potentiometer was measured. Using trigonometry, the rotation angle was estimated.

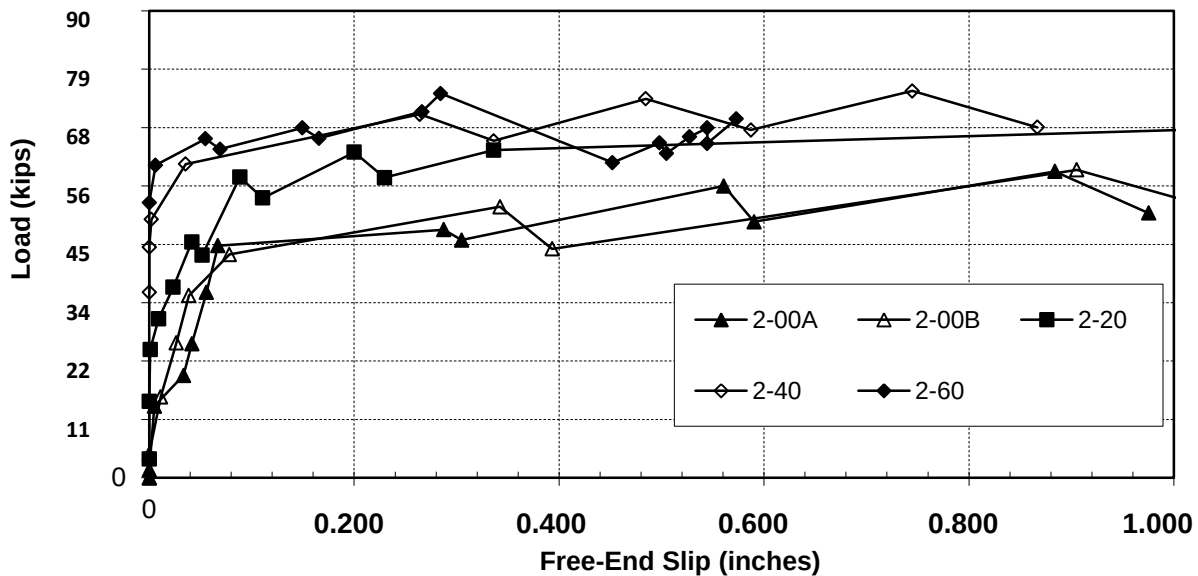


Figure 2.7 - Comparison of the Force-Slip Curves during the Pull-out Stage (24-inch Specimens)

For the three pretensioned specimens, the force drop phenomenon was more notable for the specimen with higher pretension level. By comparison with the 12-inch specimens of the same pretension force, the extent of force drop was larger. However, unlike the pretensioned 12-inch specimens, no strand rotation was detected. For the specimen 2-20, 2-40 and 2-60, the effective pretension force after the strand was released was 3, 36 and 53 kips, respectively. Unlike the pretensioned 12-inch specimens, higher pretension level caused larger effective pretension force in the strand for the 24-inch pretensioned specimens; and the effective pretension force did not decrease with time. For specimens 2-20, 2-40 and 2-60, due to the Hoyer's effect, the free end strand started to move into the concrete when the pull-out force was increased from the initial effective pretension force to 25, 50 and 60 kips, respectively.

Concrete surface strain increased with time and pull-out force. Initially, the concrete surface strain kept increasing while the effective pretension force remained the same following release of the prestressing strand. This indicates that the pretension force at transfer was a time-dependent process. In the transfer zone, the pretension force transfers to the surrounding concrete, and results in the redistribution of strain within concrete. Higher pretension force causes higher

effective pretension force and higher concrete surface strain. Later, the concrete surface strain increased with the pull-out force.

2.3.2.3 Discussion of the 36-inch Specimens

For the two 36-inch non-pretensioned specimens (3-00A and 3-00B), free slip occurred when a pull-out force of 13k was attained, and when the adhesion capacity along the specimen was exceeded. The maximum pull-out force applied to these specimens was approximately 79 kips, which was the strand breaking strength. The ultimate free-end slip for specimen 3-00A was 0.063 inches, and the slip for specimen 3-00B was 0.142 inches. **Figure 2.8** shows the behavior of this strand under load. The curve of free-end slip at the beginning was very steep. This is followed by a distinct plateau for the jacking-end slip curve after the pull-out force approximates the yield strength (72 kips) of the steel. After yielding, the strand exhibited plastic tension behavior. Additionally, it was noted that the plateau for the free-end slip curve was very short, and then a drastic force drop was noted when the strand broke at the chuck. It was observed that the strand rotation was not detected for the 36-inch specimens. Also, the strand break force was less than the strand breaking strength measured in the previous strand tension testing although these strands were from the same spool of prestressing strand.

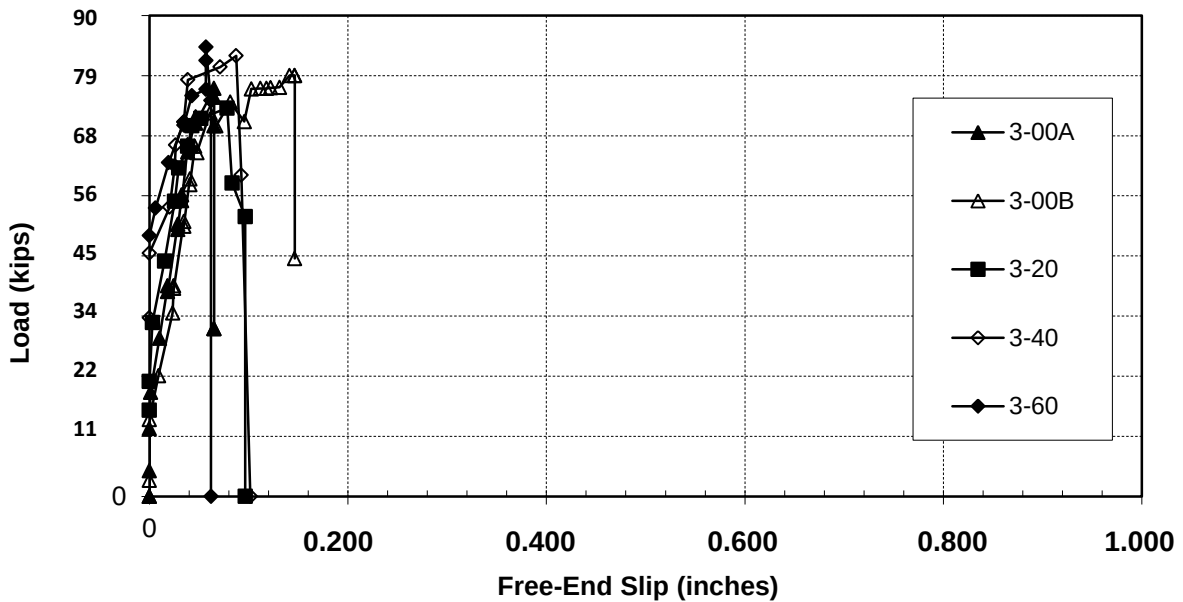


Figure 2.8 - Comparison of the Force-Slip Curves during the Pull-out Stage (36-inch Specimens)

For specimens 3-20, 3-40 and 3-60, the effective pretension force after strand release was 16, 29 and 49 kips, respectively. Higher pretension level caused larger effective pretension force in the strand for the 36-inch pretensioned specimens, and the effective pretension force did not decrease with time. For the specimen 3-20, 3-40 and 3-60, due to the Hoyer's effect, the free end strand started to move into the concrete when the pull-out force was increased from the initial effective pretension force to 21.5k, 45.5 k and 53.9 kips, respectively. All of these specimens failed by the strand breaking at relatively the same load at the chuck. The differences in failure load were likely due to differences in stress concentration at the chuck rather than the quality of the strand.

2.3.3 Analysis of Results

Figure 2.9 presents a compilation of some of the data in Table 2.1

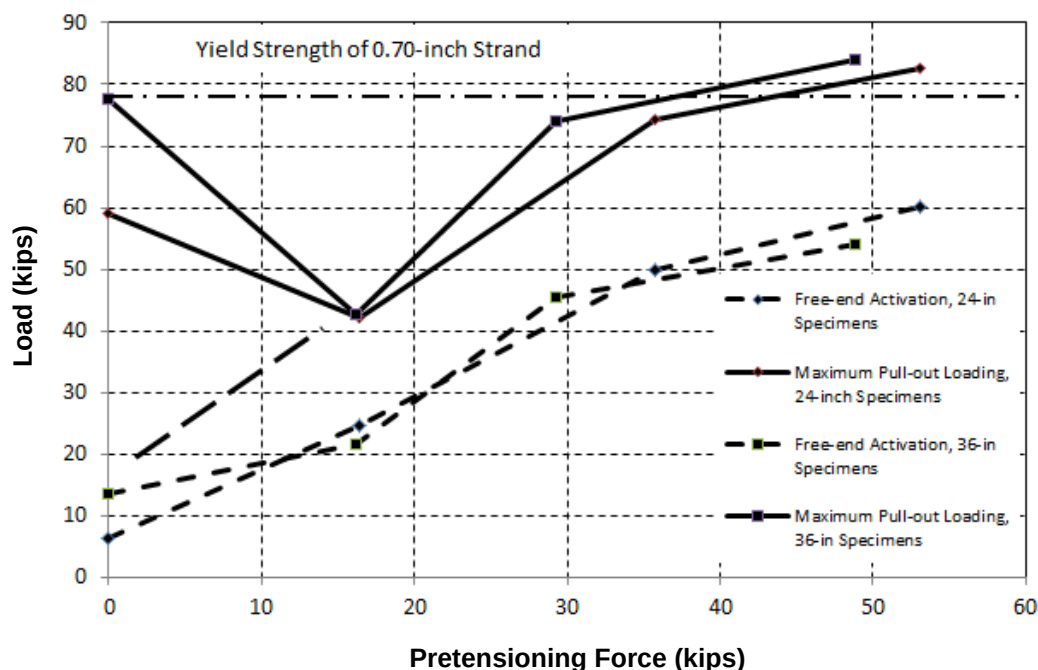


Figure 2.9 – 0.70-inch Strand Behavior with Varying Pretensioning Force

A couple of behavioral points stand out. Each specimen without a prestressed strand behaved differently than the ones with varying levels of prestressing force. The non-tensioned strand for the 36-inch specimen broke under a tensile load. The 24-inch specimen without tensioning performed better than a 24-inch specimen with minimal tensioning. The major construction difference in these two specimens was the fact that pretensioning was released by cutting the strand. This sudden change in strand shape may have broken the adhesive bond between the strand and the concrete along the full length of the specimen, leaving only a small portion of this bond to resist strand pull-out. This was particularly evident in the 12-inch specimens where pulling on the strand resulted in either the specimen splitting or the strand slipping under very little load. The long-dashed line extension is a prediction of the Hoyer effect at lower

pretensioning levels than were tested. The adhesion and Static Friction Resistance lines appear to be linear as does the Hoyer effect line prior to strand yielding.

It is interesting to note the similarities of the plots for the 24-inch and the 36-inch embedded strands. The adhesive and static friction resistance curves are relatively the same indicating that this phenomenon is more dependent upon the level of strand pretensioning than the embedded distance.

The longer embedded strand did have a higher pull-out capacity both from a free activation standpoint and from a final pull-out strength standpoint. This appears to demonstrate that while the adhesive component adds to the total pull-out resistance, the effect of releasing the pretensioning may break these initial bonds and substantially reduce the adhesive bond between the strand and the surrounding concrete. When the adhesive bond is not disturbed, adhesion alone can provide enough resistance to break a strand—even a larger diameter strand.

2.4 Modeling of Strand Behavior

2.4.1 Transfer Length Modeling

Consider the diagram shown in **Figure 2.10**.

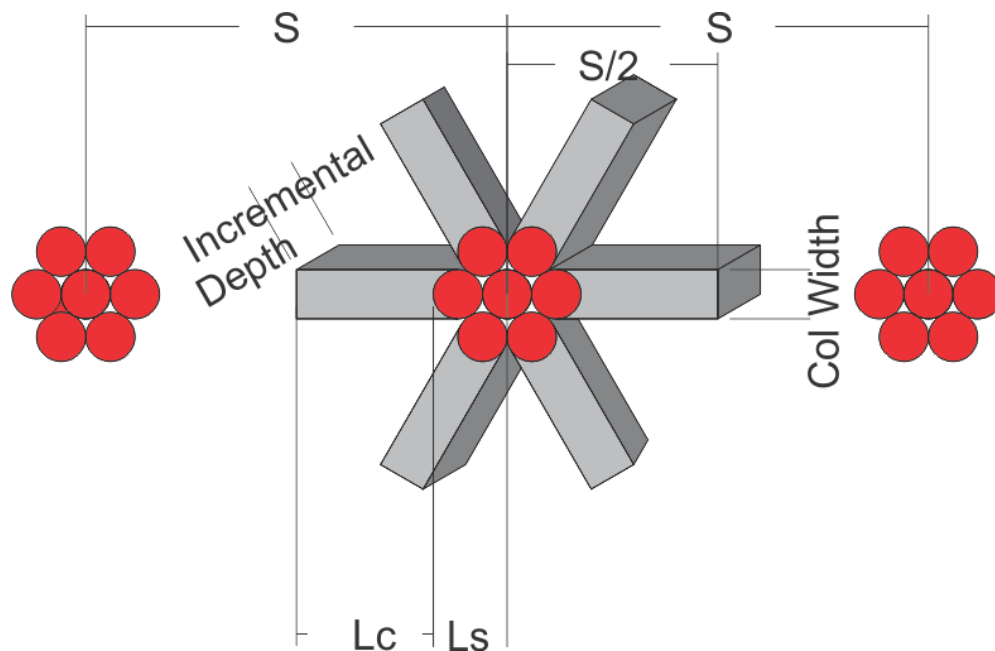


Figure 2.10 – Free-Body Diagram of Strand and Concrete Interaction Model

If a section of strand with a discrete thickness is envisioned, then using existing mechanics of materials relationships, the stress in the strand along the length of the strand can be estimated using an iterative process.

Referring back to **Equation 2.3**, the radial strain in the steel is dependent upon the initial prestressing that occurred at the time the precast beam was cast. This radial strain, when released, will be distributed to the surrounding concrete and in radial relaxation of the strand. For simplicity, it was assumed that this force was applied to concrete columns radiating out from the exterior wires of the strand. The width of the columns could be modeled as the diameter of the exterior wires and an incremental depth of these columns can be used to develop a column cross-section. In the calculations that were used to develop these models, an incremental distance along the length of the strand of 0.1-inch was used. For simplicity, all of the concrete columns were kept the same. The length of the miniature column can be described as half the strand spacing less the radius of the strand.

Another simplifying assumption was that the size of the steel was taken to be the width of the exterior wire and the same incremental depth as the concrete column was used. The length of the steel can be modeled as the radius of the strand. Using these simplifications as a basis, then three equations of equilibrium can be used to estimate the force being applied to the concrete surrounding the strand. Using **Equation 2.3**, the radial shrinkage can be estimated. This shrinkage is then released as the strand is released.

Equations 2.4 and **2.5** can then be rearranged. They can be written in terms of the pressure on the concrete and the pressure on the steel. The total force from both will equal the amount of force generated by the expansion of the strand as the longitudinal strain tries to elongate. The rearrangement of the formulas can be written as follows:

$$P_{conc} = L_{conc\ col} / \Delta_{conc\ col} A_{conc\ col} E_c, \text{ and } P_{steel\ col} = L_{steel\ col} / \Delta r_{steel\ col} A_{steel\ col} E_s \quad (2.6)$$

Also, the total length between the center of the strand and the end of the miniature column will remain the same after the strand is released. This can be expressed as follows:

$$L_{c\ initial} - \Delta_{concrete} + L_{s\ initial} + \Delta r_{strand} = L_{Total} \quad (2.7)$$

Or,

$$\Delta_{conc\ col} = \Delta r_{steel\ col}$$

One other equation of equilibrium must be satisfied. This equation deals with the relationship of the force distribution. The pressure from the elongated strand can be written as follows:

$$P_{strand\ release} = P_{conc} + P_{steel\ col} \quad (2.8)$$

The pressure from strand release can be also written as such:

$$P_{strand\ release} = \nu \varepsilon_s A_s E_s \quad (2.9)$$

Substituting **Equations 2.6** and **2.9** into **Equation 2.8** gives:

$$\nu \varepsilon_s A_s E_s = \frac{L_{conc\ col}}{\Delta r_{steel\ col} A_{conc\ col} E_c} + \frac{L_{steel\ col}}{\Delta r_{steel\ col} A_{steel\ col} E_s} \quad (2.10)$$

This is an equation with one unknown, Δr_{strand} . Solving for this variable gives:

$$\Delta r_{steel\ col} = \frac{\left[\frac{L_{conc\ col}}{\Delta r_{steel\ col} A_{conc\ col} E_c} + \frac{L_{steel\ col}}{\Delta r_{steel\ col} A_{steel\ col} E_s} \right]}{\nu \varepsilon_s A_s E_s} \quad (2.11)$$

Knowing $\Delta r_{steel\ col}$ and $\Delta r_{conc\ col}$, the pressure placed upon the concrete can be estimated using **Equation 2.6**. This would be the incremental force applied by the concrete surface to the steel. At each incremental distance along the strand, there are six exterior strands so this force can then be multiplied by 6 to get the total lateral force applied to the strand at this location.

From a strand behavioral standpoint, one of two things can then happen. The strand can either slip or not. If there is not enough residual frictional force between the section of interest and the end of the beam, then the strand will slip. The resulting change in longitudinal deflection will then become a function of the amount of resistance provided by the frictional force developed between the concrete and the strand.

The total force exerted upon each column is based upon the radial expansion of the strand. The equation for static friction is generally given in the form:

$$F_r = \mu_s F_n \quad (2.12)$$

Where,

F_r is the Static Frictional Force
 μ_s is the Static Coefficient of Friction between Surfaces, and
 F_n is the Normal Force to the Frictional Surface

This is a maximum friction force which can be applied to our system. If we consider that strand slip occurs until the system stabilizes, then this can be used as an estimate of the revisiting force.

In keeping with the model, the contact surface will be six times the width of the strand multiplied by the incremental section length. This resisting force can then be applied as a force which reduces the longitudinal strain of the strand by a specific amount in this section.

The other scenario is that there has been sufficient frictional force developed prior to reaching the section of interest to prevent slippage. That resisting force—from an accumulation of sectional frictional forces—will then keep the strand in place and the reduction of longitudinal

strain at that section will be entirely dependent upon radial expansion. The strain reduction rate for this behavior is quicker than relying upon friction only.

If a section is taken at the end of the beam, this section has a boundary condition where the residual force being applied to the strand is zero. In this section, the strand when released tries to return to its original shape, and does for the most part. As the strand is expanding, it places pressure against the concrete surface. The resistance to longitudinal expansion at this stage is very small, so the strand slips, but a small frictional resistance force is applied to the strand thereby causing a slight reduction in the radial and the increase in longitudinal strain. This reduction is then accounted in the second section by reducing the total force applied to the concrete at that section.

The frictional forces will continue to add up until a sufficient resistance is developed and the strand does not slip. At this point, the change in radial strand is not dependent upon frictional forces, but upon the clamping force due to the radial deflection of the strand. This too can be calculated and applied as the entire reduction of the system. As further sections are analyzed, the reduction of strain becomes extremely small—but theoretically never fully diminished. The transfer length can be defined by the number of sections where the difference in the original longitudinal strain applied to the strand at pretensioning and the estimated longitudinal strain in the strand reaches a predefined tolerance. This can then be multiplied by the incremental width to give an estimation of transfer length in a stand.

Using this technique, an Excel spreadsheet was used to evaluate the model. The model was used to predict the transfer length of four different strands when pretensioned to a strain of 9375 microstrain. Two different tolerance points are presented in **Table 2.2**—one at 50 microstrain and the other at 1 microstrain. Three different concrete strengths were used in this analysis. They were 6,366, 8,395, and 12,712 psi. These are the strengths of the concrete used in the small-scale development length tests described in **Section 3.0**.

Table 2.2 – Estimated Transfer Lengths of Strand

Concrete Strength (psi)	Strand Size (inches)	Estimated Transfer Length at 50 $\mu\epsilon$ (inches)	Estimated Transfer Length at 1 $\mu\epsilon$ (inches)
6,366	0.50	41.8	79.2
8,395	0.50	33.8	68.0
12,712	0.50	30.3	55.9
6,366	0.60	27.8	51
8,395	0.60	24.5	44.8
12,712	0.60	20.3	37.0
6,366	0.62	25.9	47.3
8,395	0.62	22.8	41.6
12,712	0.62	19.0	34.3
6,366	0.70	19.7	35.5
8,395	0.70	17.5	31.2
12,712	0.70	14.7	25.9

Note that this model neglects any adhesive resistance that may be present and makes an assumption that the adhesive bonds were broken at strand release.

This model also indicates that the transfer length of a strand is dependent upon the concrete strength as it predicts a shorter transfer length for strand with higher strength concrete. The transfer length is inversely proportional to strand size. The larger strand has a large contact surface area which allows a higher friction force to be developed quicker. Also, the radial expansion for the larger strand allows a greater amount of pressure to be applied to the contact surface between the strand and the concrete per unit length, thereby causing the transfer length to be shorter for the larger strand.

One thing to note in the development of this analysis method is that the change in strain from 9375 microstrain to 50 microstrain is almost the same distance as between 50 microstrain 1 microstrain.

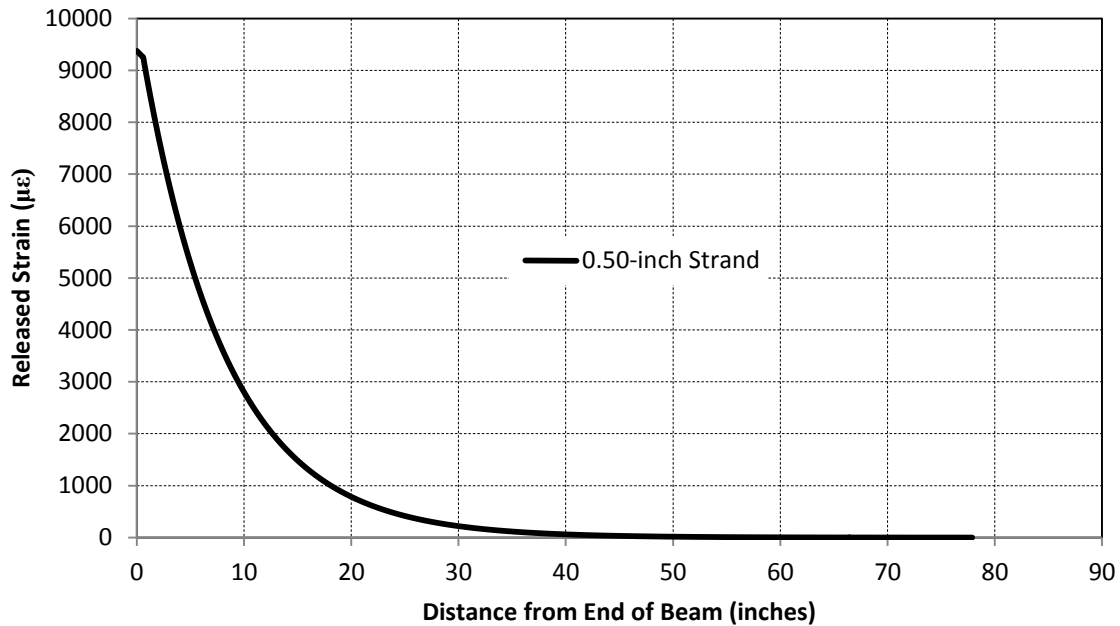


Figure 2.11 –Released Strain versus Distance from End of Beam (6,366 psi Concrete)

This information was used to create **Figure 2.11**. It could be argued that the transfer length for this specimen could be anywhere from 40 inches to 80 inches. Fifty microstrain was 0.5 percent of the released strain and was deemed to be a reasonable limit for assigning transfer length.

2.4.2 Development Length Modeling

The same general approach can be used for development length model though somewhat in reverse. In this case, the desire is to elevate the concrete-encased strand from 9,375 microstrain to 12,500 microstrain.

When pulling on strand, the strand reduces in radial dimension and elongates. The strand pulls on the concrete bond trying to resist the change in radial dimension, but if the dimension is too great, then the concrete bond will break. At that point, resistance to the elongation of the strand is created by a mechanical friction force being generated between the strand and the ruptured concrete. The strand will attempt to pull out of the concrete by twisting through the helical pattern in the embedded strand. This rotational twisting was observed during the specimen testing where bond slip failure was observed. If sufficient resistance to the rotation is maintained, then a mechanical frictional force can be developed as the strand presses against the concrete ridges at the interface. This is the same frictional resistance that was realized in the beginning of the transfer length model.

This frictional force again creates a slight change in the longitudinal strain of the strand and then reduces the applied force to the next section. The frictional forces continue until enough resistance develops to prevent the concrete bond between the strand and the concrete from

rupturing. If the bond is not ruptured, then the resistance to strand elongation is dependent upon the shear strength of the concrete. This shear strength can then be described in terms of the transfer model as the shear across the six miniature columns at each of the external wires of the strand.

Again, the development length was calculated using a small sectional procedure and an Excel spreadsheet. The calculated development length for a variety of strand sizes and concrete strengths are given in **Table 2.3**.

Table 2.3 – Estimated Development Lengths of Strand

Concrete Strength (psi)	Strand Size (inches)	Estimated Length to Elevate to 12,500 $\mu\epsilon$	Estimated Development Length (inches)
6,366	0.50	19.6	61.4
8,395	0.50	16.8	50.6
12,712	0.50	13.5	43.8
6,366	0.60	23.5	51.3
8,395	0.60	20.3	44.8
12,712	0.60	16.4	36.7
6,366	0.62	22.7	48.6
8,395	0.62	19.6	42.4
12,712	0.62	15.8	34.8
6,366	0.70	25.3	45.0
8,395	0.70	22.0	39.3
12,712	0.70	17.8	32.5

The length to elevate the strand to a strain of 12,500 microstrain is given in the third column. This value was added to the 50 microstrain transfer length to give a total development length. A plot of the level of strain from the demand point of 12,500 microstrain to 9,373 microstrain verses embedment depth is given in **Figure 2.12**. The resistance force begins gradually as the frictional force provides the resistance to strain elongation of the strand. As soon as enough resistance is developed to prevent the concrete from rupturing in tension, the shear capacity of the concrete can then be used as the resisting force. A plot of this strain reduction versus embedment depth is given in **Figure 2.13** for 0.50-inch strand and 6,366 psi concrete.

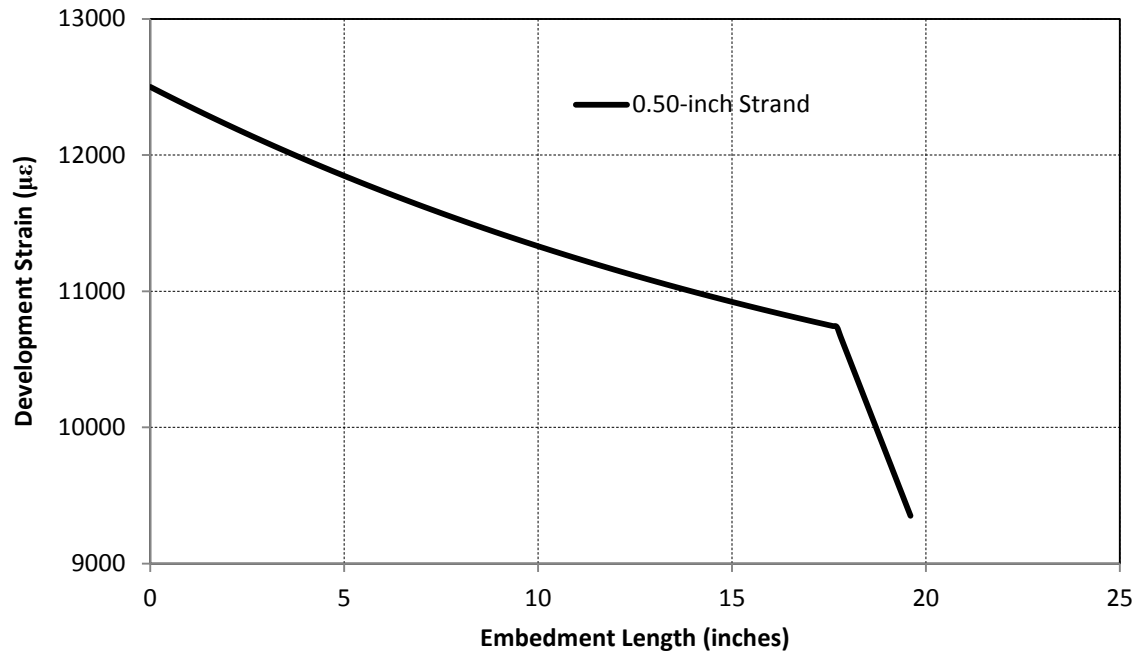


Figure 2.12 – Development Strain versus Concrete Embedment Length (6,366 psi Concrete)

The total development length becomes the sum of the transfer length and the amount of embedment length required to apply the difference in f_{ps} strain and the release strain. This model predicts that the development embedment length of the larger strand is longer than the embedment length for the smaller strand, but the overall embedment length is shorter for the larger strand.

It is counterintuitive that the development length for larger strand is shorter than that for smaller strand, but the influence of pressure between the strand and the concrete contact surface adds additional force to restrain the strand inside the concrete. **Chapter 3.0** describes the small-scale testing that was used to test this model and to measure the development length for larger diameter strand.

3.0 DEVELOPMENT LENGTH TESTING

This chapter describes the preliminary work in designing the small-scale specimens, the procedures used in testing these specimens, a summary of the individual tests, and a summary of the findings of this portion of the research.

Six specimens were produced for development length testing. **Figure 3.1** on the next page provides a schematic of these specimens. Three of these specimens were prepared with 0.7-inch, 270 ksi strand and the other three with 0.62-inch, 330 ksi strand. Each specimen was designed to be tested on each end to yield a total of 12 tests. The specimens contained a single larger diameter strand and had no shear reinforcement at the ends. This was planned so that vertical shear reinforcement would not add another variable to the test. The specimens were sized with enough concrete shear capacity (V_c) to develop a full bending capacity near the end of the specimen. One specimen for each strand size was cast with a different concrete compressive strength. Design concrete strengths of 5000, 7500, and 10,000 psi were used in this testing. Each specimen was 50 feet long and had four lifting points. A spreader bar and overhead crane were used to unload the specimens. The ends were supported when positioned before testing so that cracks were not induced into the specimens.

Though these precautions were taken, one end of one specimen was broken during handling at the lab and another end was broken during transportation. The damaged sections were cut off and the remaining portion of each specimen was tested. The tests were conducted using three-point flexural loading. Staged loading was applied at a predetermined distance from the end of the beam. The initial distance used was determined by calculating the development length using the formulas described in **Table 3.1**. The development length tests were accomplished by testing portions of the specimens with a load near the end of the specimen. This distance was changed and load applied to the specimen until the specimen either failed in flexure, shear or bond.

The objective was to attain full moment capacity with an end loading. Full moment capacity was defined as achieving an average stress over an eight-inch gauge distance of 1.25% strain. Different end positions were used until the development length distance was determined by two relatively close end loading positions, one that failed plastically in flexure and another test that failed in bond failure. The initial specimen was tested five times because the current development length formulas did not properly predict the development length distance of the larger diameter strand. As a result, initial tests were positioned too far away from the end of the beam. Some of the five tests included sections where there was vertical shear reinforcement. Though it was not desired, the additional vertical shear reinforcement did not appear to have a significant impact upon the results of these five tests.

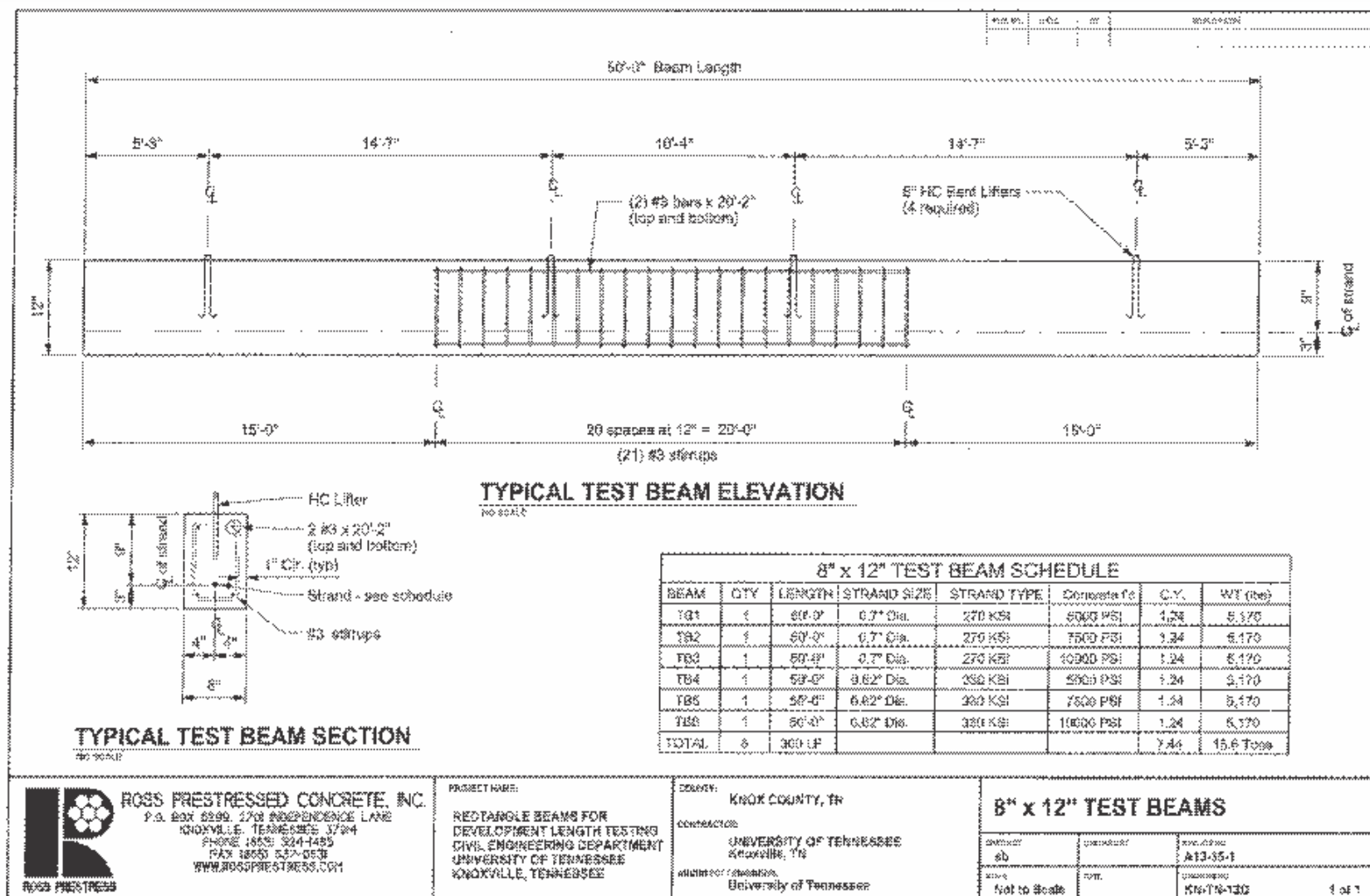


Figure 3.1 – Development Length Specimens

3.1 Estimation of Development Length using Formulas from Previous Research

During this testing the intent was to conduct two tests, one at each end of each specimen. One test was conducted to force a bond slip failure in the strand at loading and the other at a distance slightly beyond the development length. It was anticipated that the two tests would bound the

Table 3.1 - Different Proposed Equations for Development Length

Researcher	Development Length Equation
Zia and Mostafa (1977)	$l_d = \frac{1.5 f_{si}}{f'_{ci}} d_b - 4.6 + 0.18 (f_{pu} - f_{se}) d_b$
Abdalla et al. (1993)	$l_d = \left[\frac{f_{se}}{3} + 1.7 (f_{ps} - f_{se}) \right] d_b$
Mitchell et al. (1993)	$l_d = 0.33 f_{pi} d_b \sqrt{\frac{3}{f'_{ci}}} + (f_{ps} - f_{se}) d_b \sqrt{\frac{4.5}{f'_c}}$
Burdette et al. (1994)	$l_d = \left[\frac{f_{si}}{3} + 1.5 (f_{ps} - f_{se}) \right] d_b$
Buchner (1995)	$l_d = \left[\frac{f_{si}}{3} + \lambda (f_{ps} - f_{se}) \right] d_b$ $1 \leq \left[\lambda = 0.6 + 40 \epsilon_{ps} \right] \leq 2$
Lane (1998)	$l_d = \left[\frac{f_{si}}{3} + \lambda (f_{ps} - f_{se}) \right] d_b$
Barnes et al. (1999)	$l_d = \frac{5}{4} \left[\frac{f_{si}}{\sqrt{f'_{ci}}} + f_{ps} - f_{se} \right] d_b$
Shahawy (2001)	$l_d = \frac{4 f_{si}}{f'_c} d_b + \frac{6.4 (f_{ps} - f_{se})}{f'_c} d_b + 10$
Kose and Burkett (2005)	$l_d = \left[95 \frac{f_{si} (1 - d_b)^2}{\sqrt{f'_c}} \right] + \left[8 + 400 \frac{(f_{pu} - f_{si}) (1 - d_b)^2}{\sqrt{f'_c}} \right]$
Ramirez and Russell (2007)	$l_d = \left[\frac{120}{\sqrt{f'_{ci}}} + \frac{225}{\sqrt{f'_c}} \right] d_b \geq 100 d_b$
AASHTO	$l_d = \kappa (f_{ps} - 2/3 f_{se}) d_b$

The symbols used in Table 1.2 are defined as follows:

d_b = nominal strand diameter (either 0.7 or 0.62-inch)

f'_c = concrete compressive strength at 28 days (psi – See Table 1);

f'_{ci} = concrete compressive strength at time of release (psi – See Table 1);

f_{ps} = average stress in prestressing steel at ultimate load (ksi – 270 for 0.7, 330 for 0.62);

f_{pu} = ultimate tensile strength of prestressing strand (ksi – 270 for 0.7, 330 for 0.62);

f_{se} = effective stress in the prestressing steel after losses (ksi – Assumed a total reduction of stress at release to be 75% of original release strain.);

f_{si} = effective stress in prestressed steel after short-term losses (ksi – Used the same value as f_{se} .);

ϵ_{ps} = strain in prestressed reinforcement at nominal strength. (9310 microstrain for 0.7, 8921 microstrain for 0.62)

Table 3.2 – Development Length Estimations

Strand Size	0.7-inch			0.62-inch		
Release Conc. Compressive Strength (psi)	6366	7122	10,126	6366	7122	10,126
28-day Conc. Compressive Strength (psi)	6344	8395	12,712	6344	8395	12,712
Development Length by Researcher (in)						
Zia and Mostafa (1977)	37.5	33.8	24.6	34.0	30.2	20.6
Abdalla et al. (1993)	123.0	122.3	121.5	76.0	75.4	74.6
Mitchell et al. (1993)	66.9	61.9	53.0	46.0	42.9	36.2
Burdette et al. (1994)	114.2	113.5	112.5	73.0	72.5	71.9
Buchner (1995)	92.3	91.8	91.1	65.6	65.3	65.0
Barnes et al. (1999)	126.8	122.2	110.5	94.1	89.6	77.6
Shahawy (2001)	145.3	130.5	94.4	120.7	108.8	79.4
Kose and Burkett (2005)	58.5	55.7	48.0	67.6	63.9	54.5
Ramirez and Russell (2007)	90.8	84.0	72.8	80.4	74.4	64.5
AASHTO	147.6	147.1	146.4	105.0	104.6	104.0
Average (neglecting Zia and Mostafa Estimate)	107.3	103.2	94.5	80.9	77.5	69.7
80% of Average	85.8	82.6	75.6	64.7	62	55.8

actual development length of the specimen. Concrete surface strain and strand strain measurement data were collected at each load stage.

Table 3.1 gives formulas developed for development length by previous researchers. An estimation of the development length of the 0.7 and 0.62-inch strand based upon these development length formulas is provided in **Table 3.2**. Values measured and estimated for each formula variable are given below the table or previously in **Table 3.1**.



Figure 3.2 – String Potentiometer Positioned at the End of the Extended Strand

Development length estimates vary considerably from one researcher to another. Some of these equations were developed for 0.5 and 0.6-inch strand and relate the diameter of the strand and the difference in ultimate strand stress and stress at the time of loading to the development length. Some account for the difference in concrete strength, but for simplification, many ignore this parameter. As with transfer length, the three components of stress transfer—adhesion, mechanical interlock, and Hoyer’s effect—should also be incorporated into a more rational approach that can be uniformly applied to any prestressing strand size and concrete strength.

In an effort to force a strand slip failure in at least one of the development length tests, the load point was placed at a location which represented 80% average position shown in **Table 3.2** and was applied in staged increments. The increments were before cracking, at yield, just after yield and within the plastic behavior of the strand, if attained. The specimen was monitored for slip failure using physical measurements and a string potentiometer attached to the strand extending from the end as shown in **Figure 3.2**. Surface strain measurements were taken at each loading increment and the average strain was calculated for each gauge length. As the strain approached steel yield, the incremental deflection was used to determine when surface strain measurements would be taken particularly within the plastic region of the beam. **Figure 3.3** shows the typical arrangement of DEMEC points which were used to determine the average change in strain between gauge points. The intent was to examine the strain distribution within the steel across the length of the beam. Testing was discontinued when the specimen failed in either bond, shear, or flexure.



Figure 3.3 – Typical DEMEC Gauge Point Layout

After the first test, the data collected was used to determine the load point of the second test. Data collected from the first test was examined and a projected development length was re-determined. The existing development length formulas severely overestimated the development length of the 0.70-inch and the 0.62-inch strand. As a result, the first specimen was tested a total of four times before the development length was bounded. Other specimens, particularly the ones which were tested at the end of this series of tests, only required two tests to bound the development length.

3.1.1 *Beam Capacity Checks*

Tables 3.3 and 3.4 present the original loading estimates for each specimen used during the development length testing. The estimates were based upon an assumed load point, concrete strength, and the given strand size/type. They give anticipated loading for full moment capacity as defined by a concrete compressive strain at the extreme fiber of 0.003. Testing of the specimen was continued until failure. The required shear for the moment capacity is given and compared to the calculated concrete shear capacity of the specimen.

Table 3.3 – Beam Capacity Checks—0.62-inch Strand

Specimen	BM1 East	BM1 West	BM2 East	BM2 West	BM3 East	BM3 West
Strand Size	0.62	0.62	0.62	0.62	0.62	0.62
A_{ps} (inches ²)	0.2348	0.2348	0.2348	0.2348	0.2348	0.2348
f_{pu} (ksi)	330	330	330	330	330	330
f'_{c+} (ksi)	7500	7500	9000	9000	11500	11500
d_p (inches)	9	9	9	9	9	9
θ_1	0.675	0.675	0.65	0.65	0.65	0.65
c (inches)	2.104	2.104	1.837	1.837	1.455	1.455
f_{ps} (ksi)	308.4	308.4	311.1	311.1	315.1	315.1
M_n (k-ft)	50.0	50.0	51.2	51.2	52.6	52.6
V_c (kips)	14.7	14.7	15.5	15.5	17.6	17.6
Location of Load Point (inches from end)	65	81	62	78	56	70
Load to Capacity with 32' Span (kips)	11.1	9.4	11.8	9.9	13.2	11.0
Shear at Beam Short End (kips)	9.2	7.4	9.9	7.9	11.3	9.0
Is Shear Capacity Sufficient?	YES	YES	YES	YES	YES	YES

Table 3.4 – Beam Capacity Checks—0.70-inch Strand

Specimen	BM4 East	BM4 West	BM5 East	BM5 West	BM6 East	BM6 West
Strand Size	0.7	0.7	0.7	0.7	0.7	0.7
A_{ps} (inches ²)	0.2923	0.2923	0.2923	0.2923	0.2923	0.2923
f_{pu} (ksi)	270	270	270	270	270	270
f'_c (ksi)	7500	7500	9000	9000	11500	11500
d_p (inches)	9	9	9	9	9	9
θ_1	0.675	0.675	0.65	0.65	0.65	0.65
c (inches)	2.140	2.140	1.869	1.869	1.481	1.481
f_{ps} (ksi)	252.0	252.0	254.3	254.3	257.6	257.6
M_n (k-ft)	50.8	50.8	52.0	52.0	53.4	53.4
V_c (k)	14.7	14.7	15.5	15.5	17.6	17.6
Location of Load Point (inches from end)	86	107	83	103	76	94
Load to Capacity with 32' Span (k)	9.1	7.9	9.6	8.3	10.5	9.0
Shear at Beam Short End (k)	7.1	5.7	7.5	6.1	8.4	6.8
Shear Capacity Sufficient?	YES	YES	YES	YES	YES	YES

3.1.2 Measuring Devices

Several measuring devices were used during the course of this testing. Loading was monitored and recorded using the load cells within the MTS actuators. Deflection at the point of load was monitored using the vertical positioning sensors within the MTS actuator and a string potentiometer which was attached to each specimen at the load point. Other string potentiometers were used to measure the deflection of the beam at various places along the length of the beam. Concrete surface strain measurements were taken and recorded at each loading step. Strand strain was monitored using the strain gages attached to the strand when the concrete was cast.

Originally, the specimens were anticipated to be loaded at 14 feet from the end of the beam. Strain gauges were then placed upon the strand at uniformly spaced distances to this load point. These gauges were covered in epoxy to prevent water contamination when placed within the concrete. The epoxy proved to be ineffective in preventing water contamination in many cases. After the first test, it was evident that the 14-foot estimate was a poor guess of the development length. As a result, the strain gauges were not positioned properly to give valid data for this testing. Their use was discontinued after the first test and the Gilson Surface Strain Measurement system was used to attain the average strain within the concrete specimen during testing.

Before strand release, Gilson Surface Strain Measurement inserts were placed at 8-inch centers on one side of each specimen as shown in **Figure 3.4**. Initial readings of the distance between the inserts were taken and the specimens were released from the casting bed.

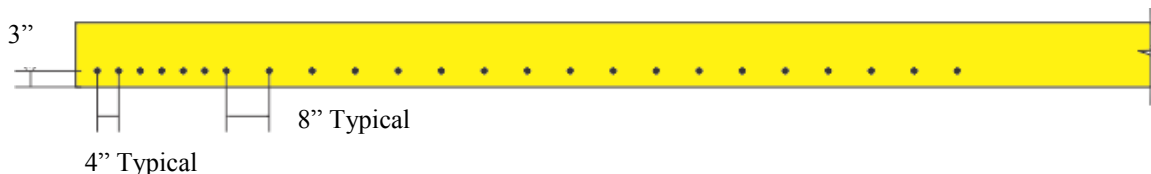


Figure 3.4 – Typical Gilson Insert Placement for each Beam End

3.1.3 Summary of Testing Procedure

Development length testing of the 0.70-inch and the 0.62-inch, collectively larger diameter strand, began in August of 2013. The specimens were cast at Ross Prestress, Inc., Knoxville, Tennessee, and Gilson inserts were placed on one side of the six specimens. These points were positioned on the side of the specimen at a distance of nine inches from the top of the specimen. The points were positioned at a distance of four inches from the end of the specimen initially and then at eight inches. An eight-inch gauge distance was used to determine average strain along that distance. After positioning, measurements were taken using a Gilson Micrometer capable of measuring distance to 0.00001 inches.

Once the initial measurements were taken, the specimens were released from the casting bed and placed at Ross Prestress' yard awaiting delivery. Each specimen was unloaded with a spreader bar which allowed picking the specimens up using the four pick points shown in **Figure 3.1**. The ends of the specimens were built without vertical steel reinforcing. This, coupled with pretensioning, made the specimens susceptible to cracking during transportation. Extreme care was given to the loading, moving, and unloading of these specimens at the precasting yard, during transportation, and at the laboratory. One end of one specimen was damaged during transportation, and the damaged end was sawn off before that specimen was tested.

Figures 3.5 shows the load frame that was used for testing. One load frame was located within 10 feet of the end of the 60-foot testing area and the other was positioned at a distance of 14 feet further in from the other load frame. Placing the load frames and actuators in this location provided a tremendous amount of flexibility in positioning the load point of the specimen.



Figure 3.5 – Load Frames with MTS Actuators at NTRC

Once the load frames were in place, the specimens were delivered. A beam ready for testing is shown in **Figure 3.6**. The other specimen was positioned in place with each end sitting on two concrete blocks to support the beams. Between the concrete blocks and the specimen a two-inch neoprene pad was used to cushion the end region of the specimen and allow for uniform distribution of the load across the cross section of the specimen. Two sheets of Teflon-coated plastic were placed on top the neoprene which allowed the specimens to expand freely without restraint under loading. Two square tube sections were bolted to the side of the concrete blocks.

These were placed to secure the specimen to the block and prevent any side rotation during testing—see **Figure 3.7**. As an additional safety precaution a metal frame was constructed and used—**Figure 3.8**. Cribbing was also placed at selected positions along the length of the beam in the event of sudden failure of the specimen.



Figure 3.6 – Positioning of Specimens at NTRC



Figure 3.7 – Live End of Development Length Testing Specimen



Figure 3.8 – Side Bracing Frame and Cribbing

Two steel blocks with a rounded side were used to load the specimen. This placed two point loads within a distance of two inches of each other. This is shown in **Figure 3.9**.



Figure 3.9 – Load Point Plating

After each specimen was in position, electronic instrumentation was installed which consisted of vertical displacement string potentiometers, strand slip monitoring string potentiometers, and embedded strain gauges. The vertical displacement string potentiometer attachment is shown in **Figure 3.10**. A hole was drilled into the specimen at the desired location two inches below the top of the specimen. After drilling, a coarse-threaded screw was inserted through the attached loop of the string potentiometer, and then the screw was placed into the hole. The string potentiometer was placed on top of a modified jack stand and affixed using duct and masking tape. The end of the string potentiometer was spring-loaded and as the beam deflected

downward, the string potentiometer sent electrical signals to the LabView equipment which indicated the change in length of the string. The string was positioned vertically plumb in all directions. The looped end of the string potentiometer had a circular insert attached to it. This allowed the rotation of the screw with vertical displacement. Other than the slight rotational error, this set up provided a very accurate accounting of the vertical displacement during the test.



**Figure 3.10 – Vertical Displacement
String Potentiometer Attachment**

Strand slip monitoring was conducted in two different ways. First, a string potentiometer was clamped to the strand—**Figure 3.11**—and screwed to the end of the beam. This instrumentation was adequate for small amounts of slip. If a large amount of slip was encountered then the strand would rotate and the measurement would be distorted by the rotation of the strand. Therefore, physical measurements were taken from a marked spot at the end of the beam to the string potentiometer clamp. These measurements were taken using a caliper and were taken at each load interval. Records of these interval measurements were recorded and copies of these data sheets are included in **Appendix C** of this dissertation.



Figure 3.11 – Strand Slip Monitoring String Potentiometer at Beam Live End

Embedded strain gauges were installed upon the strand at various places along the length of the beam. The strain gauges and the string potentiometers were wired into a LabView SCXI 1001 chassis using 1520 and 1521 modules. LabView programming was performed and electronic measurements were kept during the test and differing sampling rates depending upon the length of the procedure being performed. During loading, a one second recording rate was used and while the measurement of the distance between Gilson insert points were being made, a thirty second recording rate was used. **Figure 3.12** shows the LabView Chassis setting atop an instrumentation equipment table near the specimen being tested.

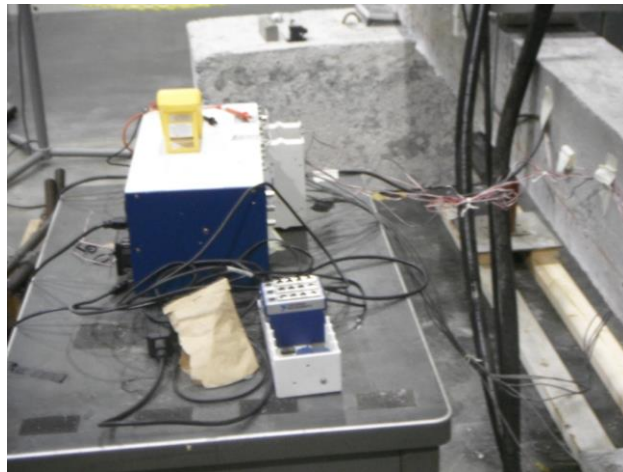


Figure 3.12 – LabView Chassis used to Record Electronic Instrumentation Measurements

The LabView programming allowed the data to be recorded within a text file which can be opened and resaved as an Excel workbook file. As mentioned, most of the data from the strain gauges was suspect and eventually the strain gauges were not used. An epoxy coating was used to protect the strain gauges during the course of the test. This was proven to be ineffective. Latex caulk was used in the full-scale beam tests and this proved to be much more reliable.

Once the load location was selected, load increments were selected. The first load point was near the calculated cracking strength of the beam, and the second near yield. The remaining points were selected in the plastic region of the specimen. With these points established, the MTS controller was programmed. Load control was used for the cracking and yield loads, and then deflection control was used to load the beam in the plastic region. The MTS equipment was programmed to record the data, and the data was then reduced for analysis.

Manual measurements for deflection at the string potentiometer locations, slippage observations, bearing pad observations were also recorded at each staged loading interval. These are included in the lab data sheet of **Appendix C**. Also, Gilson insert measurements were made and recorded upon datasheets. These were the measurement for concrete surface strain at the centroid of the strand.

Following the testing of one end of the specimen, a dust/moisture barrier as shown in **Figure 3.13** was assembled using a dining fly and four-mil polyethylene sheeting. A water sump was made by stacking 4 x 6 lumber for a water barrier and lining the floor and water barrier with polyethylene sheeting. A shop vacuum was used to remove excess water from the sump. A concrete wet saw was used to cut off the damaged portion of the specimen. Wooden cribbing was used to support the free ends of the beam following the cutting.



Figure 3.13 – Dining Fly used to Cut a Specimen

The data from the first test was evaluated. The type of failure and the load location of the previous test were used to determine the loading position of the next test. The objective was to bound the development length with one test that failed in flexural and a second which failed in bond slip without exceeding the shear capacity of the specimen. After each load step, cracks were marked and labeled then pictures were taken of the cracking patterns. Most of the

specimens cracked in a similar manner which will be described further in the report. Some of the specimens were painted prior to testing in an attempt to make the crack patterns more prominent. This yielded less than satisfactory results. The paint that was used created a film which stretched with minor cracking. As a result, small cracks were difficult to find. The unpainted side was used to find the crack and the cracks were traced to the painted side for labeling. **Figure 3.14** shows how the crack pattern was labeled after staged loading. **Figures 3.15** shows crack development as the specimens were being tested.



Figure 3.14 – Labeling of Crack Patterns during Testing



Figure 3.15 – Crack Pattern Collage after Testing

3.2 Results of Individual Development Length Tests

A total of 17 development length tests were performed. Each specimen was given a distinct specimen number. The number consisted of three parts. The first part was a sequential letter to indicate the order of each test on the specific specimen. The letter ‘A’ was assigned to the first test, “B” to the second and so forth. The second part identifies the type of strand used in the specimen—either 0.70-inch 270ksi or 0.62-inch 330ksi strand. The third number indicates the design concrete compressive strength used in preparing the specimen.

The sections that follow describe the length of the specimen and the loading geometry being used to load the specimen. A photo is given at failure for each specimen. String potentiometer measurements were taken at the support and also at quarter points. A load versus load point deflection curve is given for each specimen tested and a strand strain vs stage loading plot is also provided.

3.2.1 Specimen A0.70-6

A0.70-6 was the first specimen tested with 0.70-inch strand and a design concrete strength of 6,000 psi. Based upon the preliminary calculations a load point of 98 inches from the end of the specimen (93 inches from the center of the support) was selected. A 2-inch thick, 10-inch wide neoprene pad was placed at each support. The specimen was 50 feet long with the distance from the center of the live end support to the dead end of 40 feet. String potentiometers were placed at 10.42, 20.42, 30.42, 42.0, and 50.0 feet from the live end of the specimen.

The initial cracking load was calculated to be 1.93 kips and f_{ps} moment was estimated to be 6.88 kips. From this staged loading points of 4, 6.5, 7, and 7.5 kips were selected. As the loading reached 7.9 kips the displacement capability of the actuator was exceeded. Additional shims were added and the specimen was reloaded. Upon reloading, the specimen broke at 7.5 kips with a deflection of approximately 8 inches. **Figure 3.16** shows the failure mode of the specimen. This failure was categorized as a flexural failure with the concrete crushing and a fully yielded reinforcing strand. No slipping of the strand was observed during this test.



Figure 3.16 – Failure Cracking of Specimen A0.70-6 at Load Point

Figures 3.17 and 3.18 show load vs deflection and load vs concrete surface strains at the elevation of the strand. The load versus deflection diagram shows the full development of the plastic behavior of the strand. The beam deflected from two to eight inches without any appreciable increase of the loading.

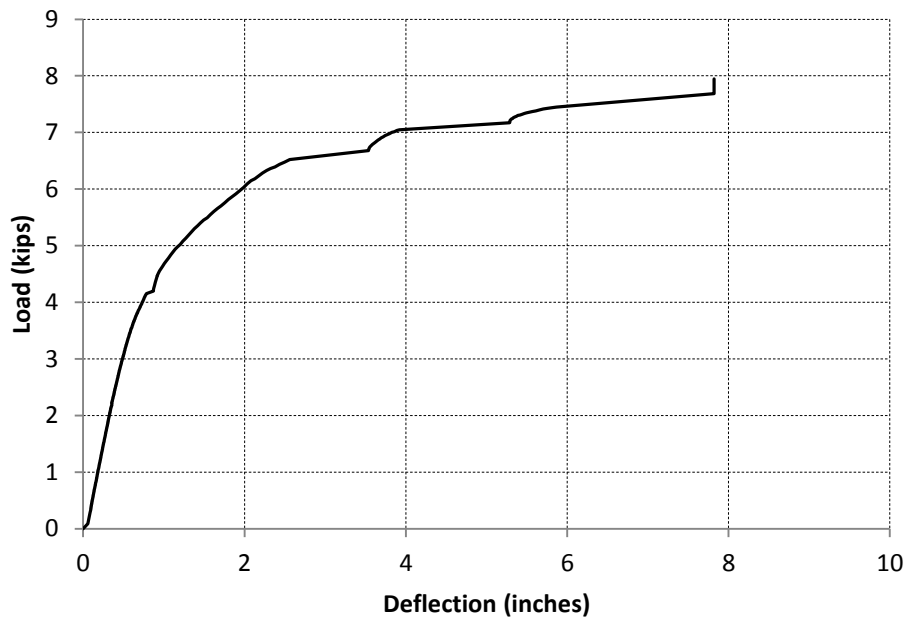


Figure 3.17 – Load versus Deflection at Load Point, Specimen A0.70-6

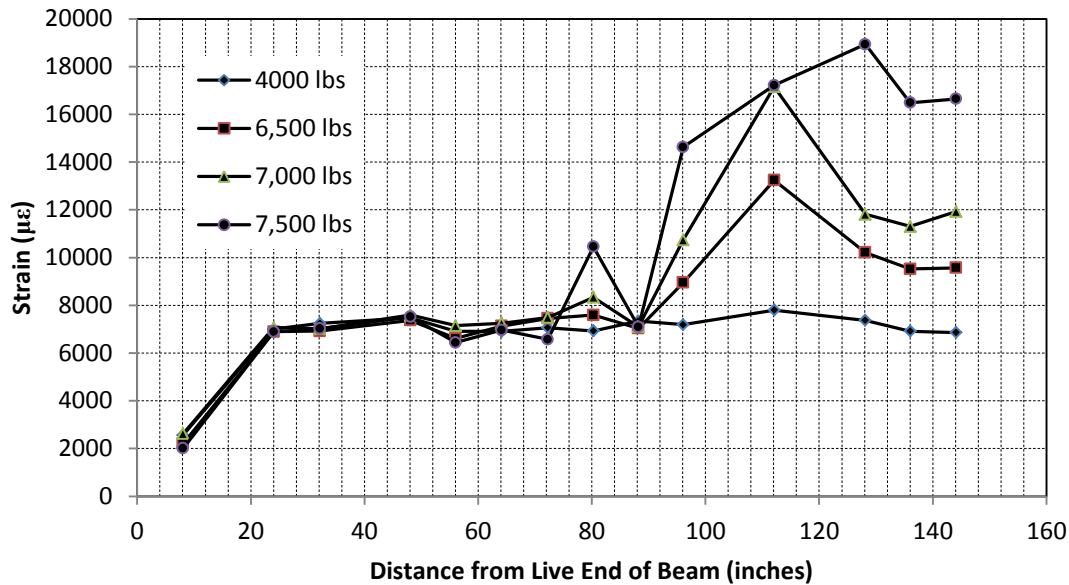


Figure 3.18 – Steel Strain versus Distance from Live End of Beam, Specimen A0.70-6

To achieve a full development length, the strand would be required to stretch beyond its yield strain (f_{ps}). The f_{ps} strain using the 2% offset method was 1.25% strain or 12,500 microstrain. As seen in **Figure 3.18**, the strain in the strand exceeded the 12,500 microstrain limit. This confirms the flexural failure observed during testing. Because f_{ps} was exceeded without strand slip, the actual development length of this strand using the given concrete strength was found to be less than the predicted 98-inch distance from the end.

Figure 3.19 indicates that there was very little strand strain response under loading from the end of the specimen to about 70 to 80 inches. Under the largest load, f_{ps} strain was achieved at a distance of 92 inches from the end. This would indicate that the embedment required to raise the strain from the release strain to f_{ps} strain would be between 12 and 22 inches. If this were added to the approximate transfer length of 24 inches, then this indicates that the development length is between 36 and 46 inches.

3.2.2 Specimen B0.70-6

This test used the same specimen as A0.70-6. A load point of 30 inches from the end of the specimen was selected for this test. The size of the neoprene pad was reduced to 6 inches at the live end, though the 10 inch pad was utilized at the dead end support. The specimen was 24 feet long with the distance from the center of the live end support to the dead end of 23.33 feet. String Potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 11 kips and f_{ps} moment was estimated to be 24.9 kips. The staged loading points of 10, 15, 20, 22, and 24 kips were selected. This specimen failed at a load of 17.9 kips as a shear crack developed in the end zone of the beam. After this crack formed, the strand began to slip. **Figure 3.19** shows the failure mode of the specimen. This failure was categorized as a shear/bond slip failure. Full moment capacity was not achieved before cracking in the end zone reduced the capability of the beam to carry load.



Figure 3.19 – Failure Cracking of Specimen B0.70-6 at Load Point

Figures 3.20 and **3.21** present load vs deflection and load vs concrete surface strains at the elevation of the strand.

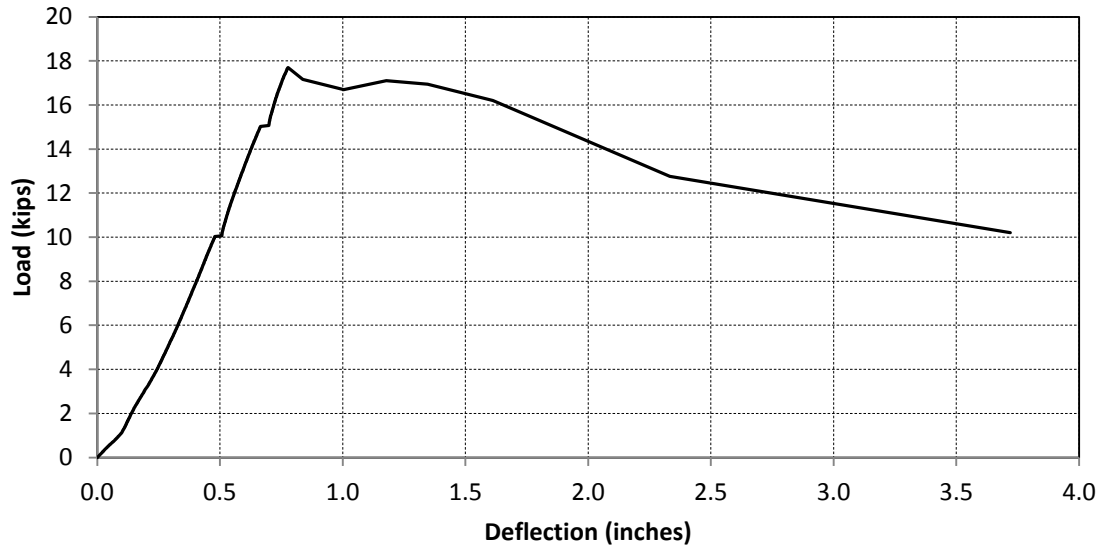


Figure 3.20 – Load versus Deflection at Load Point, Specimen B0.70-6

The maximum strain achieved during this test was 8792 microstrain. This confirms the shear/bond slip failure observed during testing. Because f_{ps} was not attained without strand slip, the actual development length of this strand using the given concrete strength is greater than 30 inches.

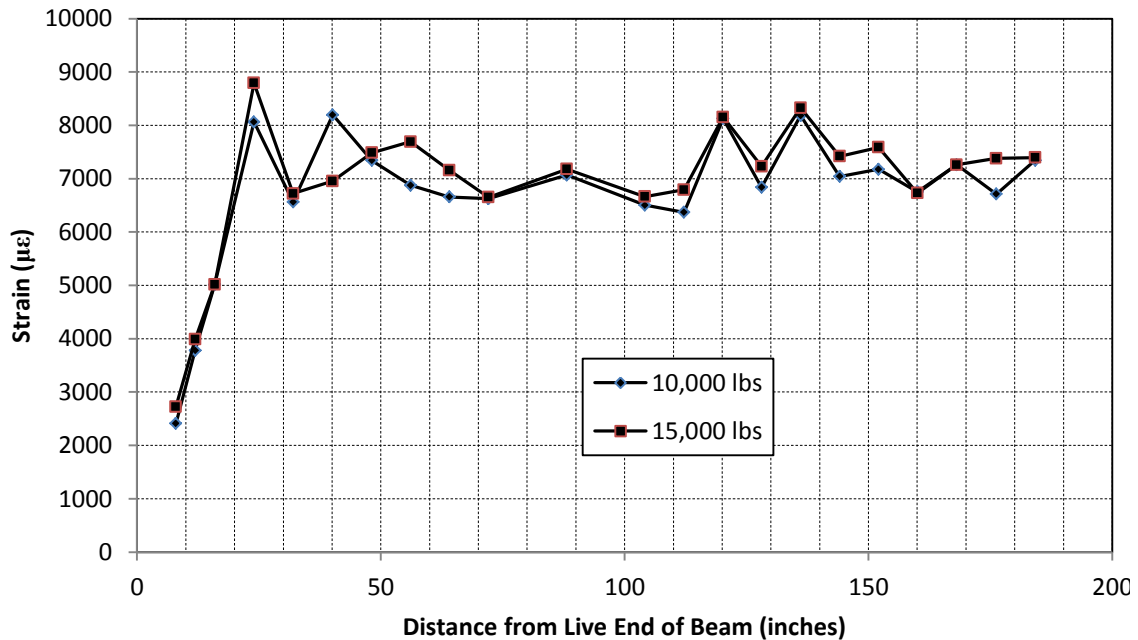


Figure 3.21 – Steel Strain vs Distance from Live End of Beam, Specimen B0.70-6

3.2.3 Specimen C0.70-6

This test used the same specimen as A0.70-6. A load point of 48 inches from the end of the specimen was selected for this test. A 10-inch neoprene pad was used at each support. The specimen was 35 feet long with the distance from the center of the live end support to the dead end of 34.17 feet. String Potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 5 kips and f_{ps} moment was estimated to be 18.8 kips. From this, staged loading points of 6, 10, 15, 17.5, 20, and 22 kips were selected. This specimen failed in flexure. **Figure 3.22** shows the failure mode of the specimen. Full moment capacity was achieved before the concrete crushed, strand yielded, and specimen failed.



Figure 3.22 – Failure Cracking of Specimen C0.70-6 at Load Point

Figures 3.23 and **3.24** present load vs deflection and load vs concrete surface strains at the elevation of the strand. The plastic moment was reached at a load of about 24.0 kips.

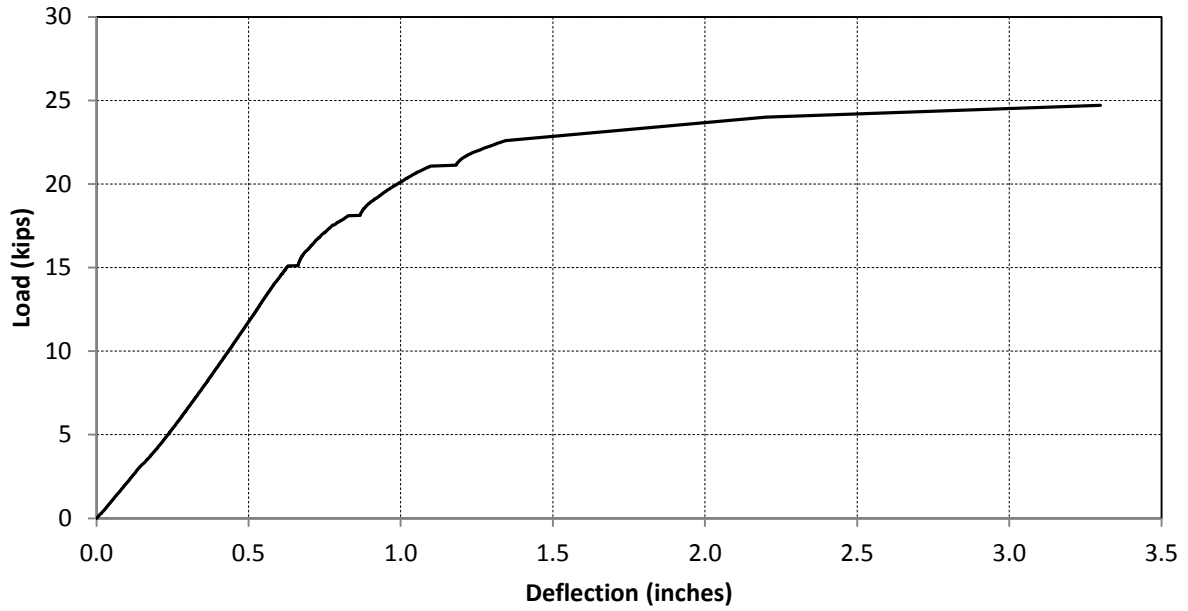


Figure 3.23 – Load versus Deflection at Load Point, Specimen C0.70-6

The maximum strain achieved during this test was 38,119 microstrain. This confirms that flexural failure was observed during testing. Because f_{ps} was attained without strand slip, the development length was less than 48 inches from the end of the specimen.

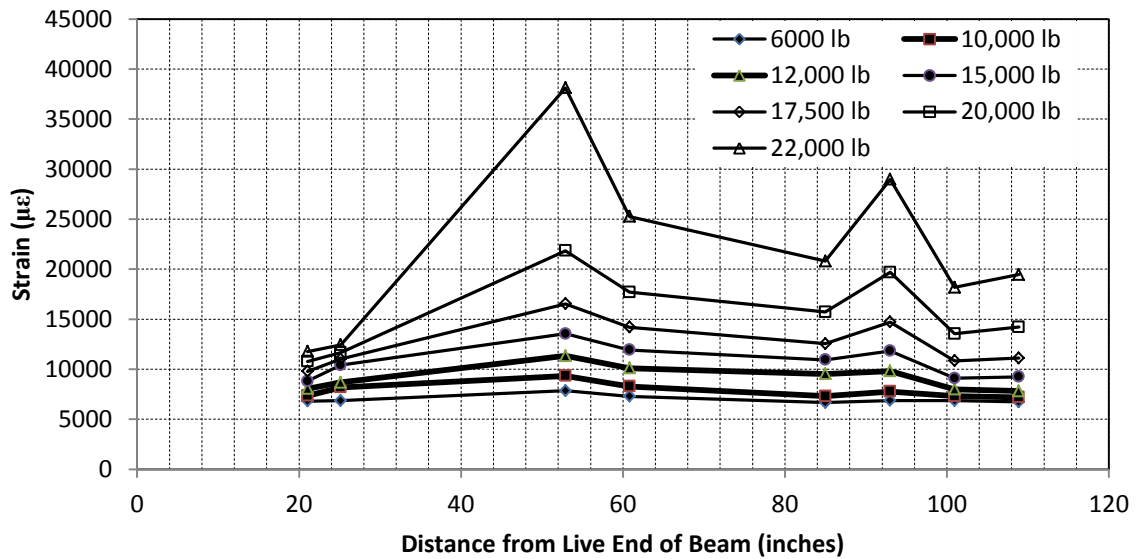


Figure 3.24 – Steel Strain vs Distance from Live End of Beam, Specimen C0.70-6

3.2.4 Specimen D0.70-6

This test used the same specimen as A0.70-6. A load point of 43 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 16.97 feet long with the distance from the center of the live end support to the dead end of 16.30 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 8.1 kips and the f_{ps} moment was estimated to be 25.4 kips. From this, staged loading points of 8, 12, 16, 20, and 21 kips were selected. This specimen failed in flexure. **Figure 3.25** shows the failure mode of the specimen. The first flexural crack occurred with a load of 8 kips. At 21 kips, the beam began behaving plastically and surface strain measurements were taken. A constant deflection rate was then used to load the specimen to failure. A shear crack formed at the underside of the specimen at 24" and extended to 35". Once this cracked formed the specimen failed suddenly. A clear thrust crack formed as a result of the combination of shear and flexure forces applied to the specimen. This crack became angulated as it reached the load point. A second crack along the bottom of the beam then formed and the concrete at the bottom of the specimen spalled off.



Figure 3.25 – Failure Mode of Specimen D0.70-6

An average steel strain of 16,500 microstrain was measured near the load point. This specimen failed at a load of 21.3 kips. No slip in the strand was observed during testing.

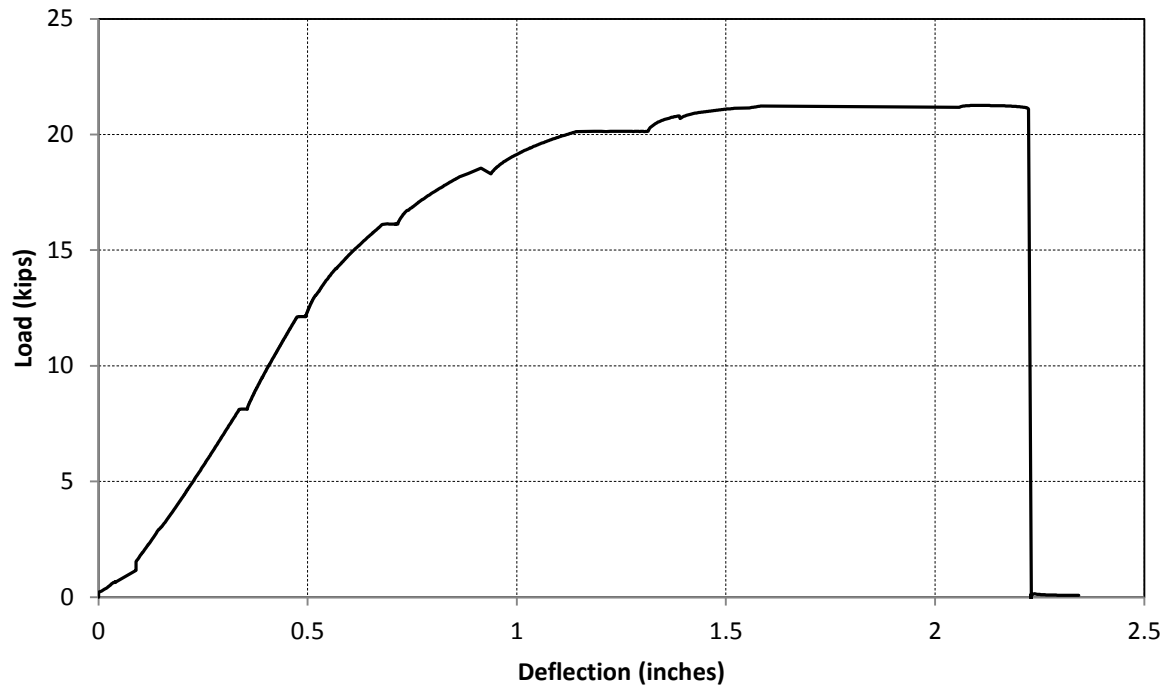


Figure 3.26 - Load versus Deflection at Load Point, Specimen D0.70-6

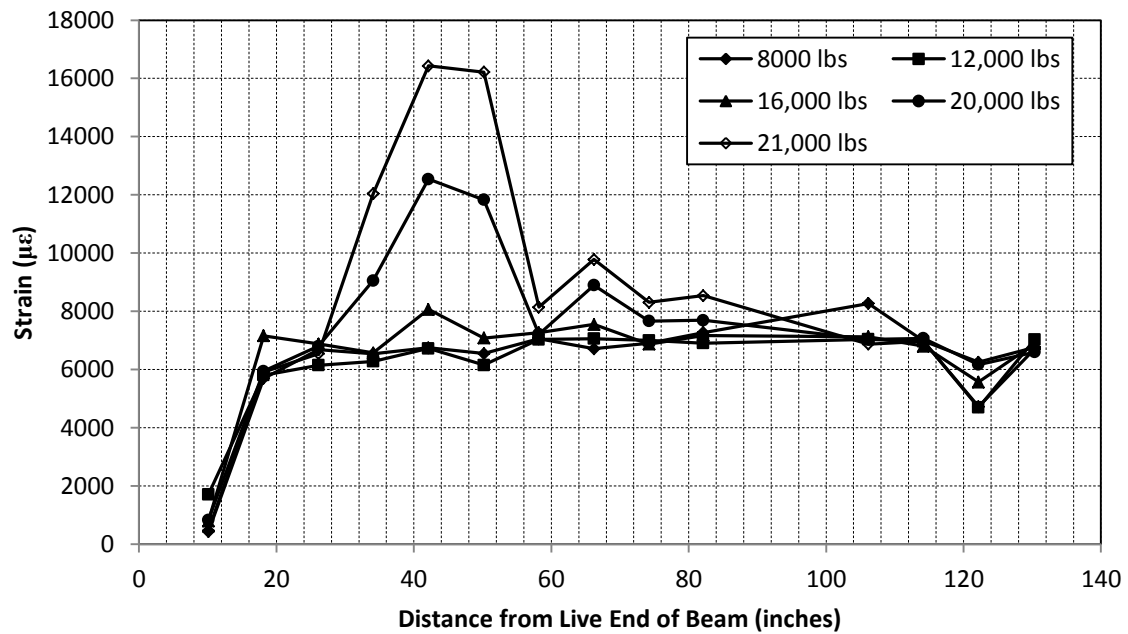


Figure 3.27 – Steel Strain versus Distance from Live End of Beam, Specimen D0.70-6

A load - deflection curve is shown in **Figure 3.26**. As the steel reached its plastic limit continued deflection occurred until the specimen broke as a result of a flexural failure. The concrete at the top of the specimen crushed and the specimen collapsed. The spalling of the bottom concrete and the crushing of the top concrete occurred simultaneously as the beam collapsed. Since the strand did not slip, this indicates that the development length is less than 43 inches.

3.2.5 Specimen E0.70-6

This test used the same specimen as A0.70-6. A load point of 37.5 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at dead end. The specimen was 13.17 feet long with the distance from the center of the live end support to the dead end of 12.5 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 9.9 kips and f_{ps} moment was estimated to be 30.8 kips. From this, staged loading points of 12, 21, 24, 27, 30, and 32 kips were selected. This specimen failed when a combination shear and flexure crack formed at a loading of 26.1 kips. **Figure 3.28** shows the failure mode of the specimen. Full moment capacity was almost achieved before slip occurred and the specimen collapsed.

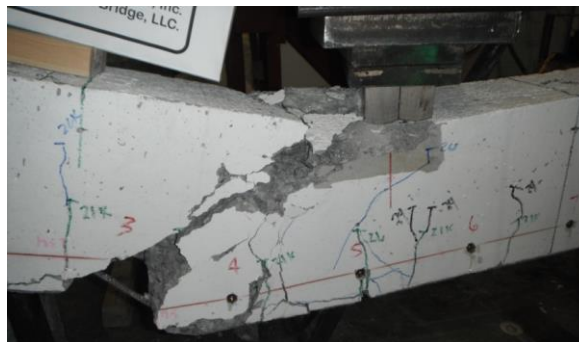


Figure 3.28 – Failure Cracking of Specimen E0.70-6 at Load Point

Figures 3.29 and **3.30** present load vs deflection and load vs concrete surface strains at the elevation of the strand.

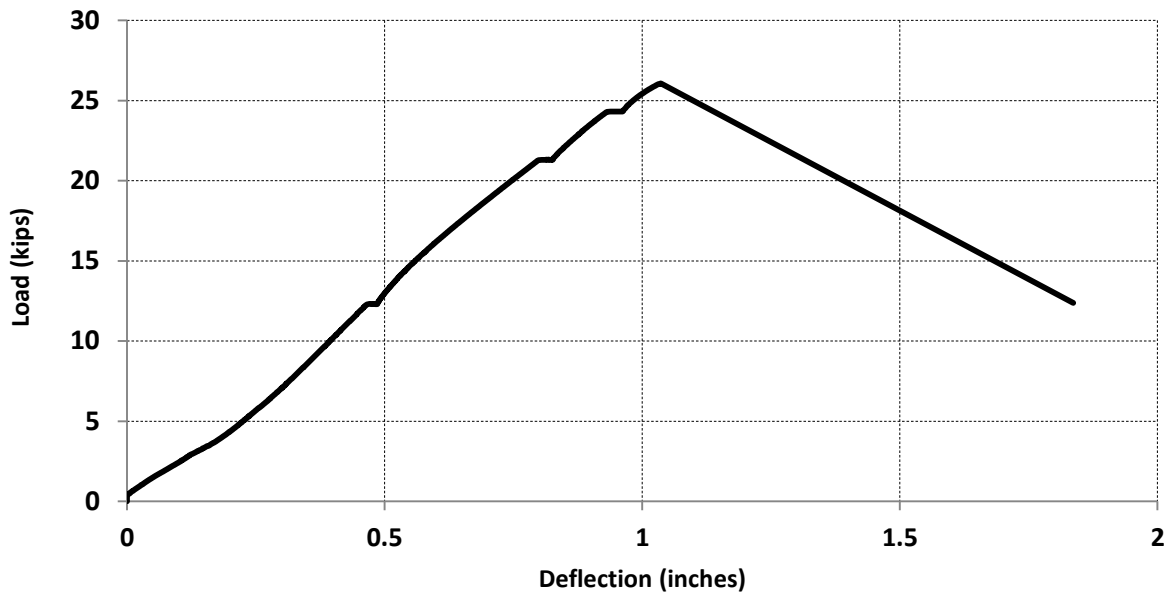


Figure 3.29 – Load versus Deflection at Load Point, Specimen E0.70-6

The maximum strain achieved during this test was 12,183 microstrain or 97 percent of the strain corresponding to f_{ps} . Because f_{ps} was not attained without strand slip, the actual development length is slightly greater than 43 inches. Linear interpolation of this specimen's results would indicate that the development length is about 44.5 inches.

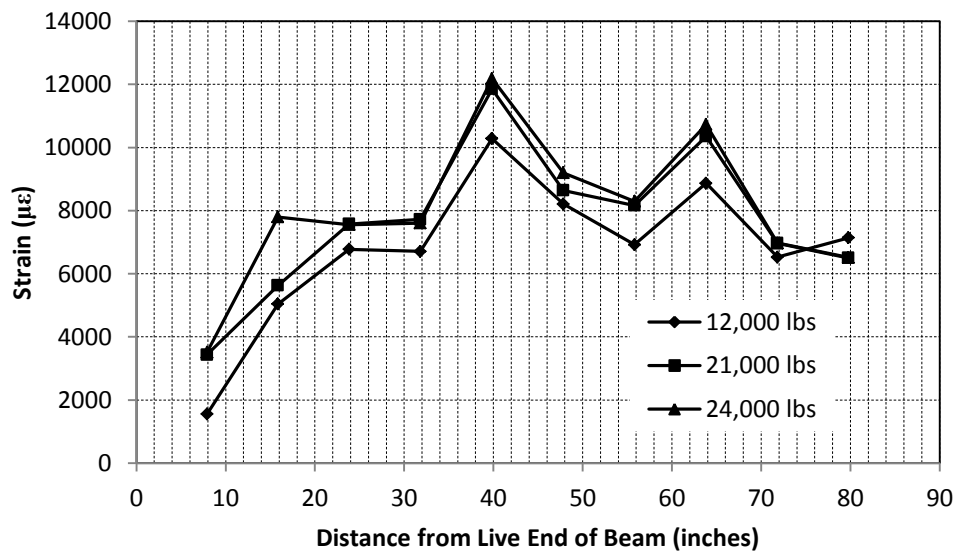


Figure 3.30- Steel Strain versus Distance from Live End of Beam, Specimen

3.2.6 Specimen A0.70-8

Specimen A0.70-8 had 0.70-inch strand and a design concrete strength of 8,000 psi. During placement, the specimen was damaged. The specimen was shortened to 41.5 feet and supported on each end. A load point of 34 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 41.5 feet long with the distance from the center of the live end support to the dead end of 40.64 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 7.6 kips and f_{ps} moment was estimated to be 20.4 kips. From this, staged loading points of 7, 11, 15, 18.5, 21, and 23 kips were selected. This specimen failed in flexure without strand slip.

Figure 3.31 shows the failure mode of the specimen. Full moment capacity was achieved in this test. Since this specimen was shortened, vertical shear reinforcement was located 40 inches from the load point. Due to the difference of the stiffness, a plastic hinge formed at the location where the stiffness changed. This specimen had an average steel strain of 11,782 microstrain near the load point and 21,170 microstrain at the hinge point. This specimen failed at a load of 23.3 kips. No slip in the strand was observed.



Figure 3.31 – Failure Mode of Specimen A0.70-8

Figures 3.32 and 3.33 present load vs deflection and load vs concrete surface strains at the elevation of the strand.

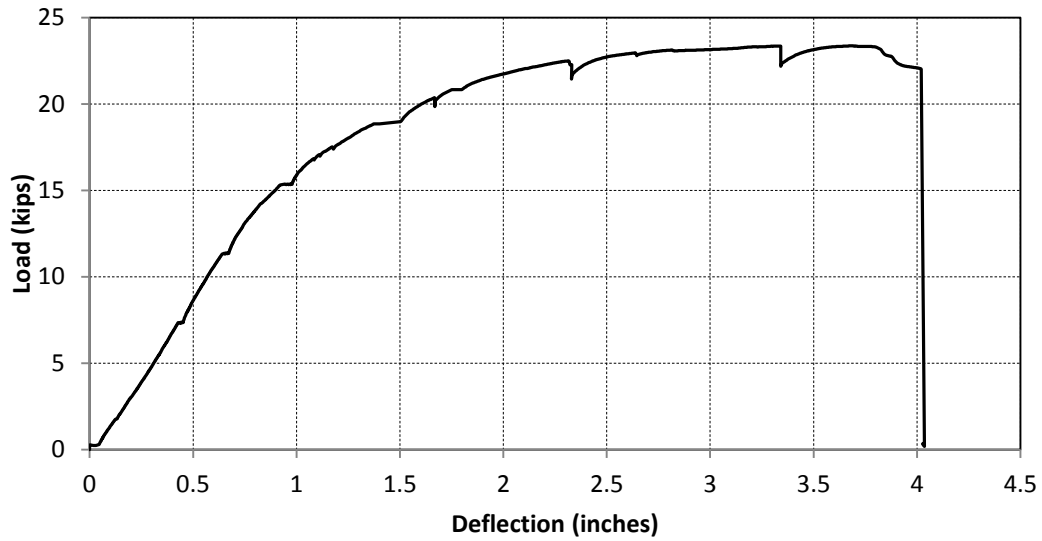


Figure 3.32 - Load versus Deflection at Load Point, Specimen A0.70-8

Since the strand did not slip, this indicated the development length is less than 34 inches.

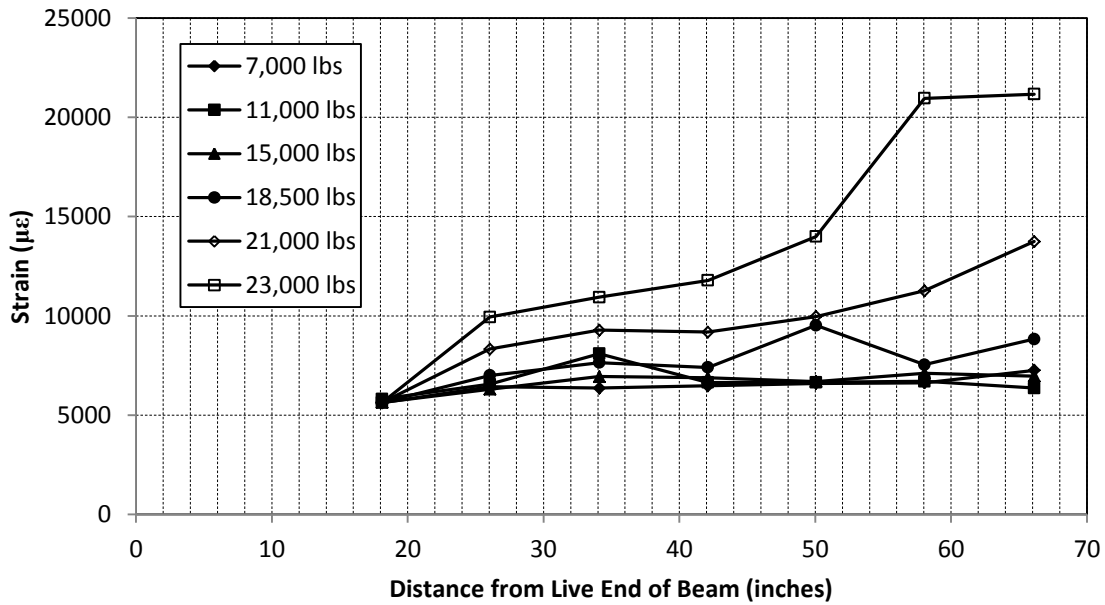


Figure 3.33 – Steel Strain vs Distance from Live End of Beam, Specimen A0.70-8

3.2.7 Specimen B0.70-8

The B0.70-8 test used the same specimen as A0.70-8. A load point of 24 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 30.17 feet long with the distance from the center of the live end support to the dead end of 29.5 feet. String Potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 11.6 kips and f_{ps} moment was estimated to be 28.7 kips. From this, staged loading points of 11, 18, 21, 23, and 25 kips were selected. This specimen failed in shear. **Figure 3.34** shows the failure mode of the specimen. Full moment capacity was not achieved in this test. As the shear crack formed on the underside of the specimen, the strand began to slip and the specimen began to collapse. This occurred at a loading of 27.9 kips. Even though the strand slipped, the beam held a residual loading of 15.1 kips. The combination flexural and shear crack formed first and then a second shear crack developed at strand slip failure. This specimen had a maximum average steel strain of 7,800 microstrain near the load point of the specimen.

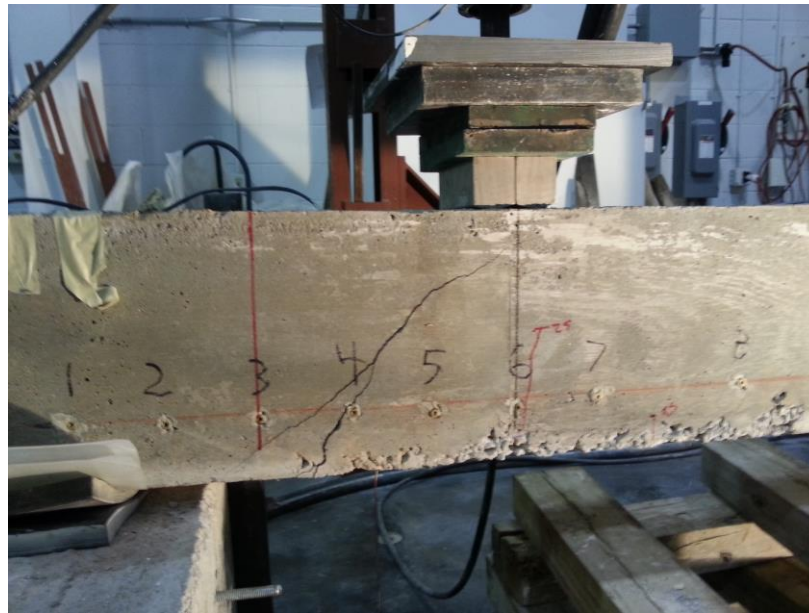


Figure 3.34 – Failure Mode of Specimen B0.70-8

Figures 3.35 and **3.36** present load vs deflection and load vs concrete surface strains at the elevation of the strand.

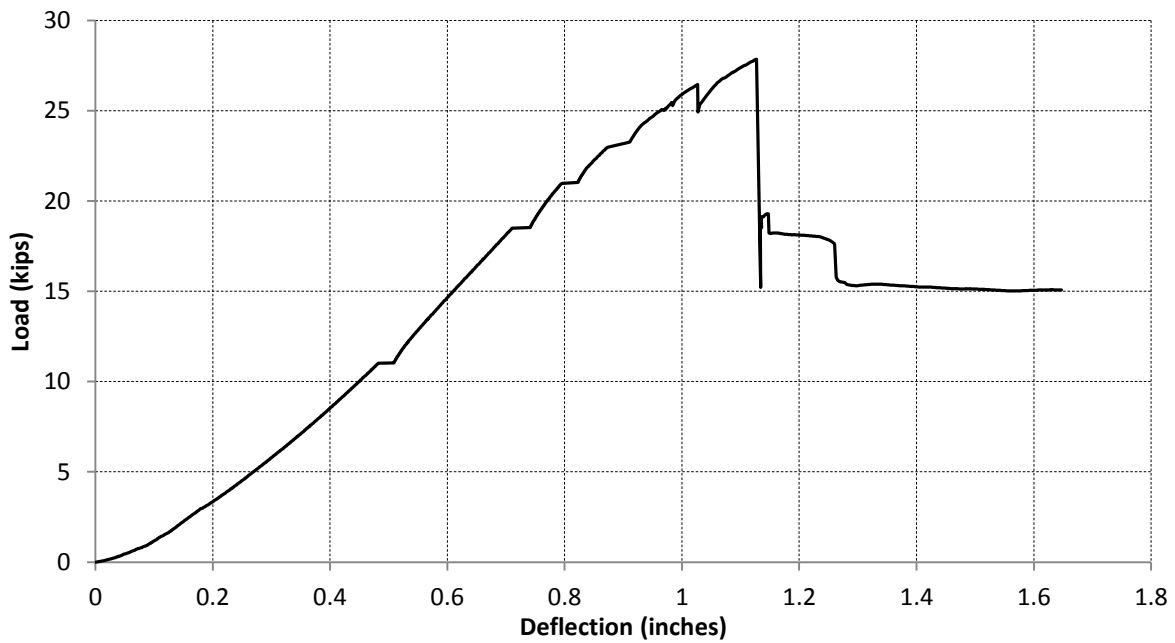


Figure 3.35 – Load versus Deflection at Load Point, Specimen B0.70-8

Since the strand slipped, this indicates that the development length was greater than 24 inches.

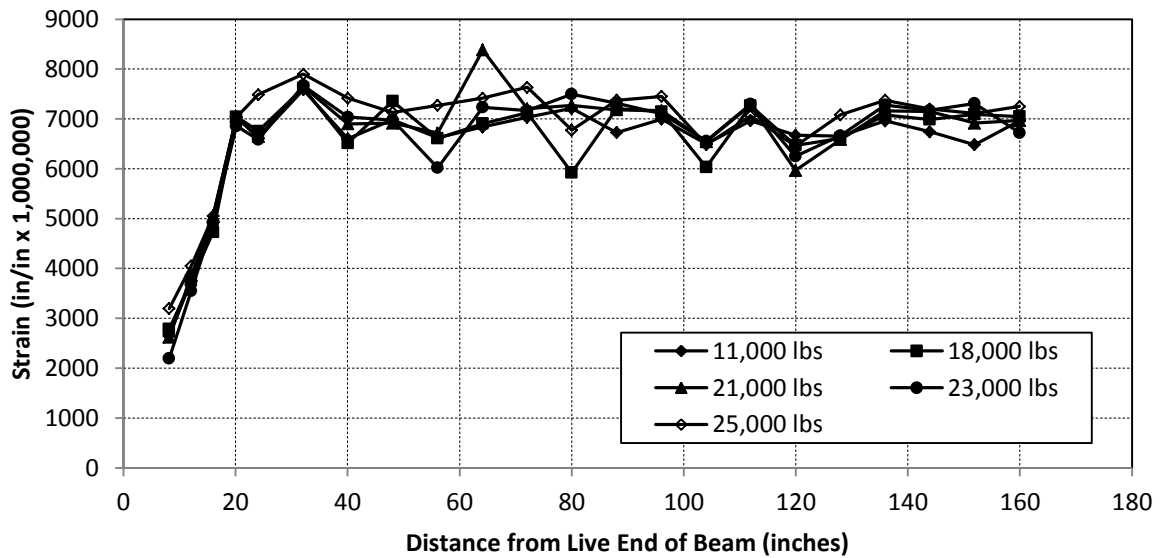


Figure 3.36 – Steel Strain vs Distance from Live End of Beam, Specimen B0.70-8

3.2.8 Specimen A0.70-10

Specimen A0.70-10 had 0.70-inch strand and a design concrete strength of 10,000 psi. A load point of 30 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 50-foot long with the distance from the center of the live end support to the dead end of 43.54 feet. String Potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 8.9 kips and f_{ps} moment was estimated to be 21.2 kips. From this, staged loading points of 9, 13, 17, 20, and 20.7 kips were selected. This specimen failed in flexure. **Figure 3.37** shows the failure mode of the specimen. A diagonal shear crack formed at 20 kips and at 20.7 kips a slip of 0.2 inches was observed. No further loading was applied. An average steel strain of 32,101 microstrain near the load point of the specimen was measured.



Figure 3.37 – Failure Mode of Specimen A0.70-10

Figures 3.38 and **3.39** present load vs deflection and load vs concrete surface strains at the elevation of the strand. There was no plastic behavior observed though f_{ps} moment was achieved.

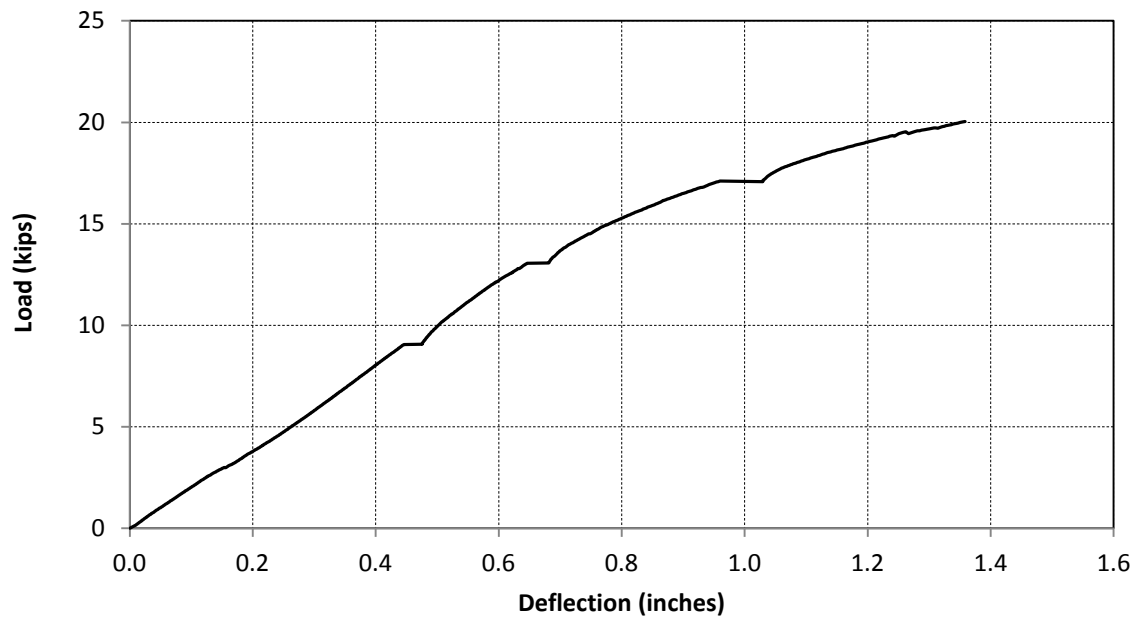


Figure 3.38 - Load versus Deflection Curve at Load Point, Specimen A0.70-10

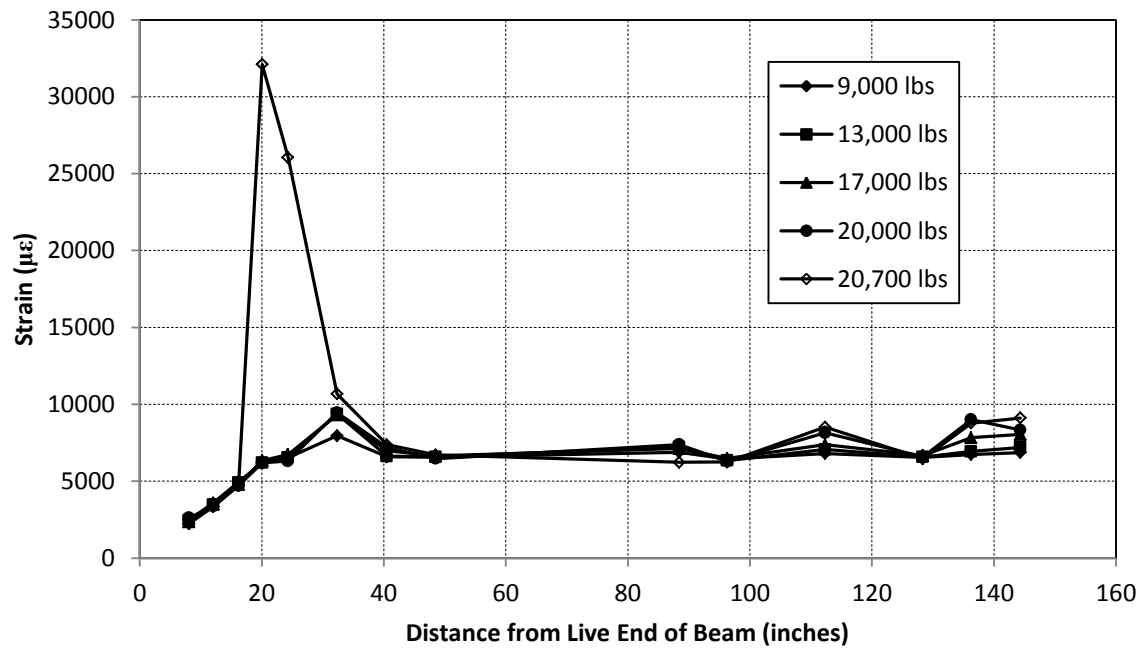


Figure 3.39 – Steel Strain vs Distance from Live End of Beam, Specimen A0.70-10

The behavior of this specimen, with a load point of 30 inches, was on the border of a slip and flexure failure. As a result, a 33-inch load spacing was selected for the next test. The testing of this specimen indicated that development length was slightly larger than 30 inches.

3.2.9 Specimen B0.7-10

This test used the same specimen as B0.70-10. A load point of 33 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 28.0 feet long with the distance from the center of the live end support to the dead end of 27.17 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 9.3 kips and f_{ps} moment was estimated to be 24.6 kips. From this, staged loading points of 15, 18, 21, 22.5, 24, and 24.7 kips were selected. This specimen failed in flexure. **Figure 3.40** shows the failure mode of the specimen. Full moment capacity was achieved in this test. The first flexural crack occurred with a load of 18 kips. At 22.5 kips, the beam began acting plastically. An average steel strain of 20,550 microstrain was attained near the load point. This specimen failed at a load of 24.7 kips. No slip in the strand was observed.



Figure 3.40 – Failure Mode of Specimen B0.70-10

Figures 3.41 and **3.42** present load vs deflection and load vs concrete surface strains at the elevation of the strand.

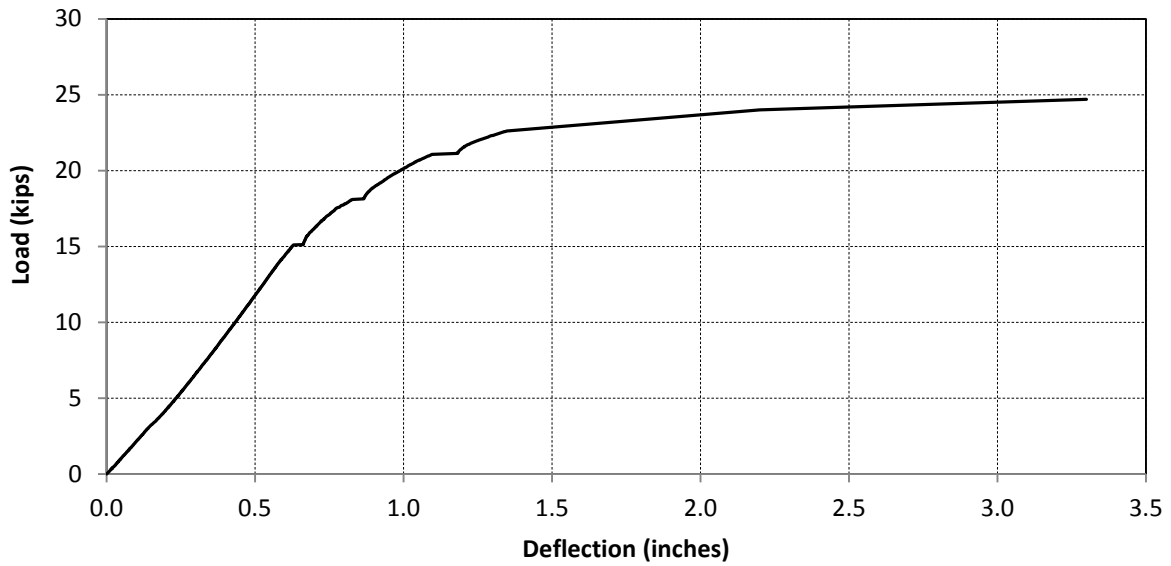


Figure 3.41 - Load versus Deflection Curve at Load Point, Specimen B0.70-10

Since the strand did not slip, this indicates that the development length is less than 33 inches.

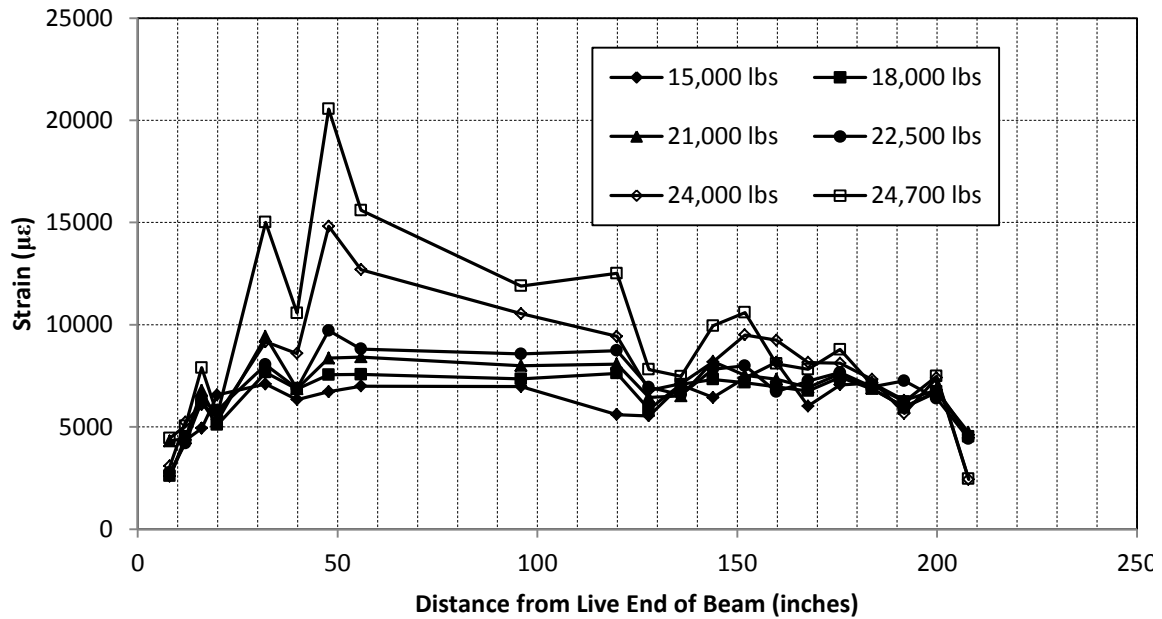


Figure 3.42 – Steel Strain vs Distance from Live End of Beam, Specimen B0.70-10

3.2.10 Specimen A0.62-6

Test A0.62-6 used 0.62-inch strand and a design concrete strength of 6,000 psi. A load point of 72 inches from the end of the specimen was selected for this test. The size of the neoprene pad was 10 inches at the live end and 10 inches at the dead end. The specimen was 39.83 feet long with the distance from the center of the live end support to the dead end of 39.0 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 5.2 kips and f_{ps} moment was estimated to be 14.1 kips. From this, staged loading points of 3, 8, 10, 12, and 15 kips were selected. This specimen failed in bond slip. **Figure 3.43** shows the failure mode of the specimen. Full moment capacity was not achieved in this test. The first flexural crack occurred with a load of 6 kips. This crack continued to expand during the entire length of the test. A bond slip failure occurred at a load of 8.7 kips and the specimen failed to take any additional load.



Figure 3.43 – Failure Mode of Specimen A0.62-6

This specimen had an average steel strain of approximately 10,500 microstrain near the load point of the specimen. This was within 84 percent of 12,500 microstrain, but did not achieve the f_{ps} strain amount.

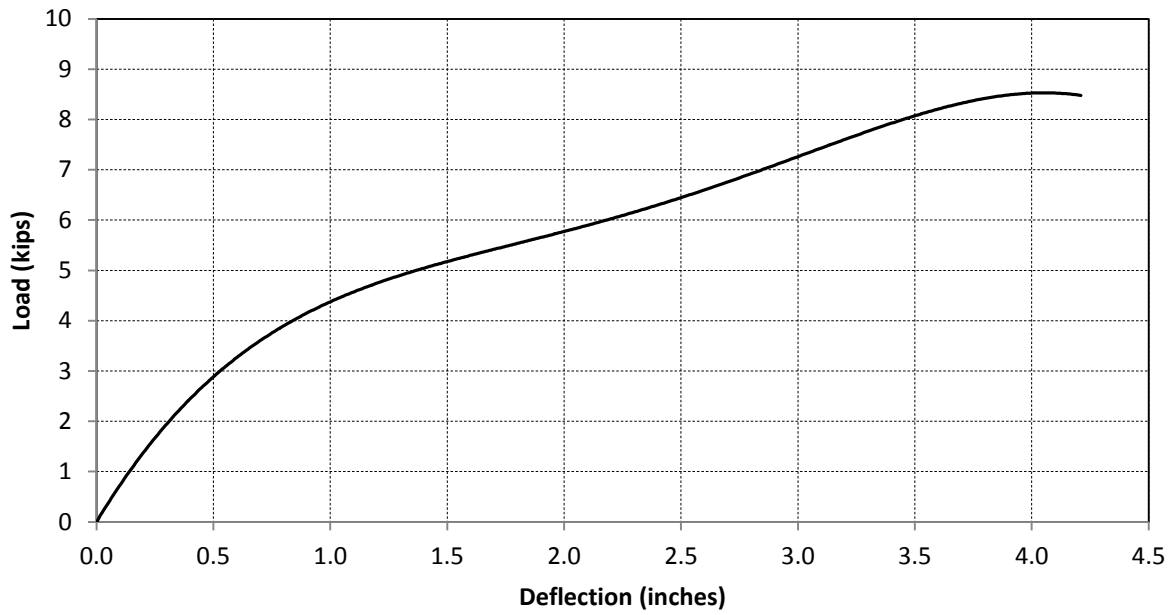


Figure 3.44 - Load versus Deflection at Load Point, Specimen A0.62- 6

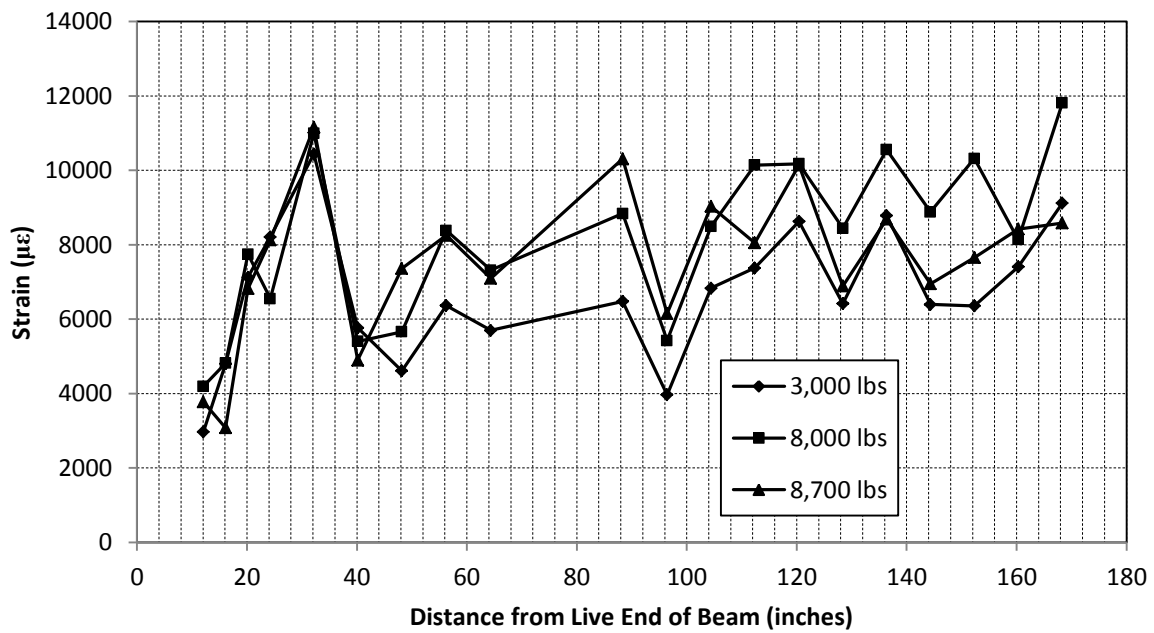


Figure 3.45 – Steel Strain vs Distance from Live End of Beam, Specimen A0.62-6

The loading of the beam was discontinued when the strand began to slip at maximum loading. Based upon this test the development length is slightly longer than 72 inches.

3.2.11 Specimen B0.62-6

When preparing to test Specimen B0.62-6, the specimen broke during the set up and as a result, this end of the specimen was not tested. The damaged portion was cut off leaving a remaining portion of 20.79 feet. Since the load point of the previous test was 72 inches, it was decided to load this specimen in the middle. Ideally, the specimen would have been better loaded at a shorter distance, but there was a necessity to bound the development length at this distance and this loading. Therefore, the next test C0.62-6, was to be loaded in the middle at 10.35 feet from the end.

3.2.12 Specimen C0.62-6

The C0.62-6 test used the same specimen as A0.70-6. A load point of 10.42 feet from the end of the specimen was selected for this test. The size of the neoprene pad was 10 inches at the live end and 10 inches at the dead end. The specimen was 20.79 feet long with the distance from the center of the live end support to the dead end of 19.96 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 3.6 kips and f_{ps} moment was estimated to be 11.8 kips. From this, staged loading points of 2, 4, 8, 10, and 12 kips were selected. This specimen failed in flexure at a load of 12.9 kips. **Figure 3.46** shows the failure mode of the specimen. Full moment capacity was achieved in this test. This was one of the first tests run for these specimens and strain measurements were not taken at 12.0 kips.



Figure 3.46 – Failure Mode of Specimen C0.62-6

The MTS LVDTs and load cells were used to obtain load and deflection data, but the programming did not work on this test. This data was reproduced using physical measurements and deflection measurements from the recorded LabView string potentiometer data. At 12.0 kips, it was observed that the beam was acting plastically. The concrete crushed as a flexure cracked developed below the load point. The average steel strain at 10.0 kips for this specimen was 8888 microstrain near the load point of the specimen. This is shown in **Figure 3.47**. Though this does not demonstrate that f_{ps} strain was achieved, the reconstructed load versus deflection diagram demonstrates this behavior. No slip in the strand was observed.

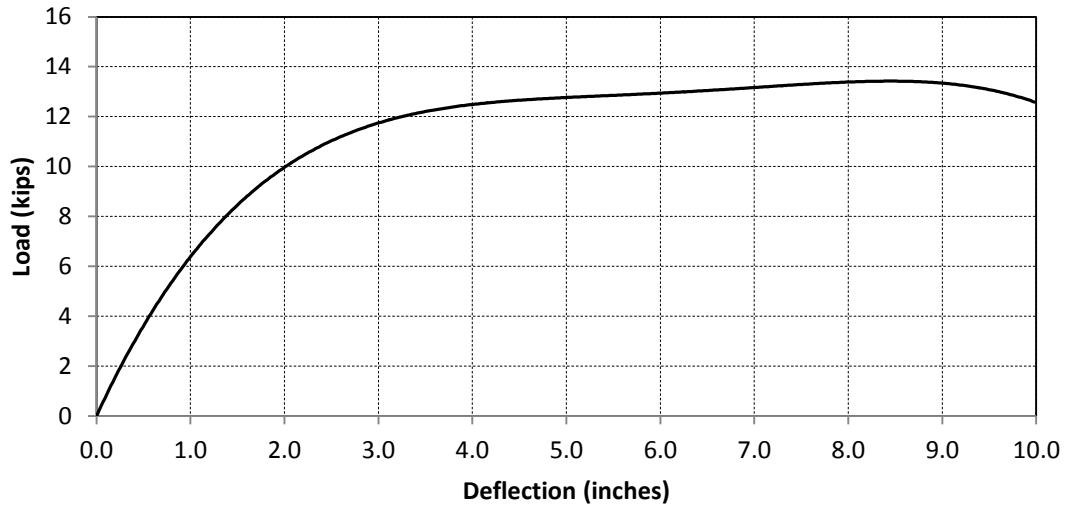


Figure 3.47 - Load versus Deflection at Load Point, Specimen C0.62-6

The load versus deflection curve was plotted and is presented in **Figure 3.47**. As the steel reached its elastic limit continued deflection occurred until the specimen broke as a result of a flexural failure. The concrete at the top of the specimen crushed and the specimen could not resist higher loading. Since the strand did not slip, this indicated that the development length is less than 10.98 feet.

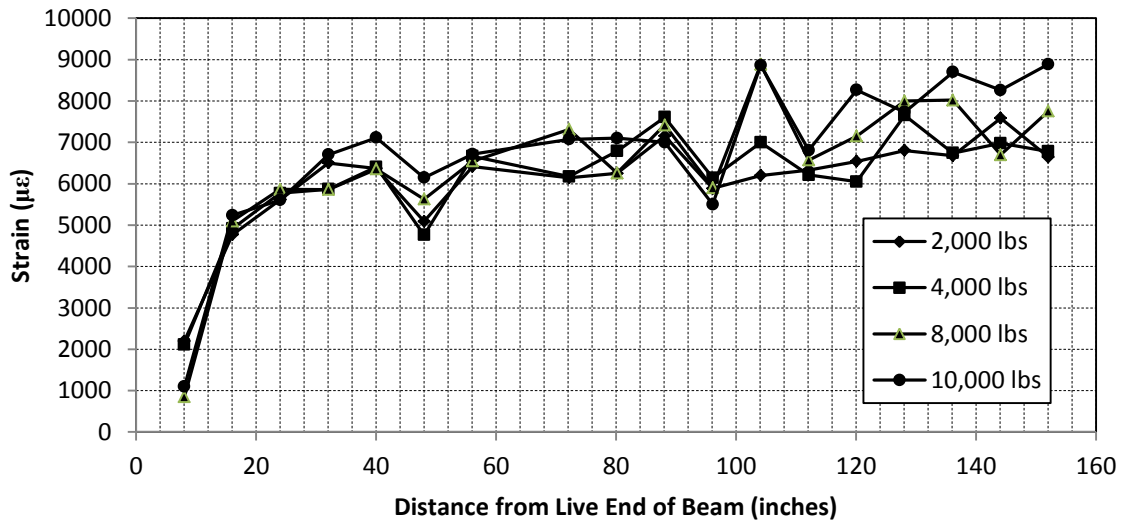


Figure 3.48 – Steel Strain vs Distance from Live End of Beam, Specimen C0.62-6

3.2.13 Specimen A0.62-8

Specimen A0.62-8 had 0.62-inch strand and a design concrete strength of 8,000 psi. A load point of 48 inches from the end of the specimen was selected for this test. The size of the neoprene pad was 6 inches at the live end and 10 inches at the dead end. The specimen was 39.83 feet long with the distance from the center of the live end support to the dead end of 39.0 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand. This specimen was damaged during transportation and the end was cut off prior to testing.

The initial cracking load was calculated to be 5.2 kips and f_{ps} moment was estimated to be 14.1 kips. From this, staged loading points of 6, 10, 14.5, and 15 kips were selected. This specimen failed in flexure. **Figure 3.49** shows the failure mode of the specimen. Full moment capacity was achieved in this test. The first flexural crack occurred with a load of 12 kips. This crack continued to 14.5 kips and as this loading level was attained the beam began acting plastically and surface measurements were taken. A shear/flexural crack formed at the underside of the specimen. Once this crack formed, the specimen failed suddenly and the specimen gave way. A clear thrust crack formed as a result of the combination of shear and flexure forces. This crack became angulated as it reached the load point. A second crack along the bottom of the beam then formed and the concrete at the bottom of the specimen spalled off. A final load of 15.9 kips was attained before failure. An average steel strain of 16,373 microstrain near the load point of the specimen was attained. This specimen failed at a load of 15.9 kips. Slip was observed in the specimen following the formation of the flexural/stress cracks.



Figure 3.49 – Failure Mode of Specimen A0.62-8

A load versus deflection curve is shown in **Figure 3.50**.



Figure 3.50 - Load versus Deflection at Load Point, Specimen A0.62-8

As the steel reached its elastic limit, continued deflection occurred until the specimen broke as a result of a flexural failure. The concrete at the top of the specimen crushed and the specimen collapsed. The spalling of the bottom concrete and the crushing of the top concrete occurred simultaneously as the beam collapsed. The strand did slip as cracking caused by flexural cracking promulgated within the end zone of the specimen. This indicates that the development length was less than 48 inches.

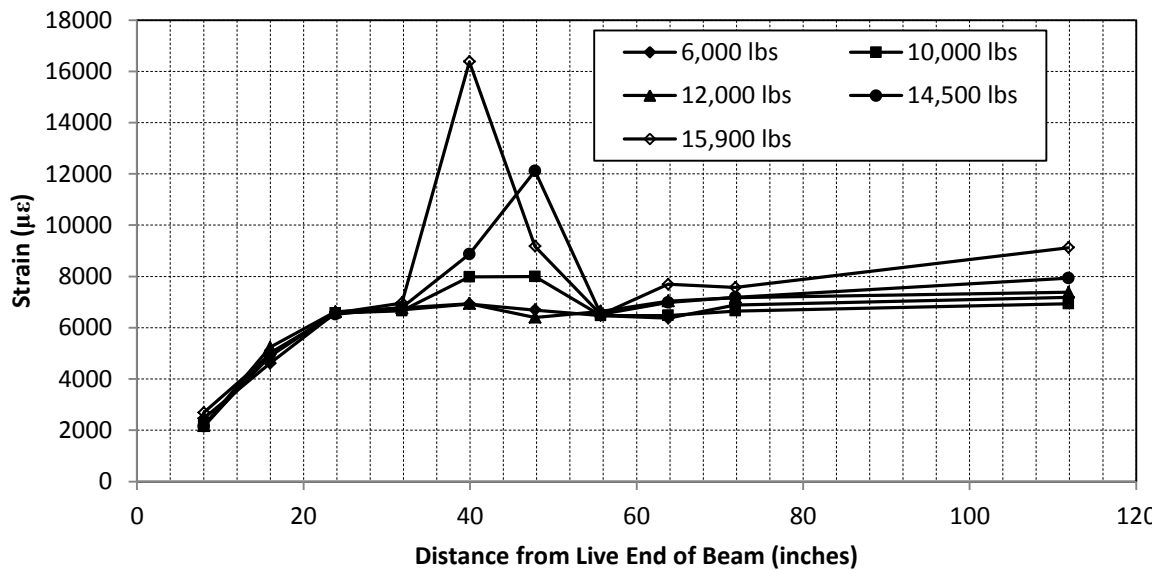


Figure 3.51 – Steel Strain versus Distance from Live End of Beam, Specimen A0.62-8

3.2.14 Specimen B0.62-8

The Specimen B0.62-8 test used the same specimen as A0.62-8. A load point of 40 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 21.46 feet long with the distance from the center of the live end support to the dead end of 20.79 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

The initial cracking load was calculated to be 7.21 kips and f_{ps} moment was estimated to be 24.6 kips. From this, staged loading points of 10, 15, 20, 23, and 25 kips were selected. This specimen failed in bond slip. **Figure 3.52** shows the failure mode of the specimen. This specimen was loaded to 10 kips and measurements were taken. When loading the specimen to the second stage, a single flexural crack appeared below the load point at a loading of 12.9 kips. At this point, the strand began to slip and a residual load of 12.9 kips was all that the specimen would hold. This was well less than the calculated value and, compared to the other tests, was very premature.



Figure 3.52 - Failure Mode of Specimen A0.62-8

Only the strain measurements for the 10.0 kips loading were recorded and they are plotted in **Figure 3.53**. The strain level recorded at the load point was 6933 microstrain. This test was believed to be inconclusive, and the data from it were not used. The strand may have not been bonded properly when the concrete was vibrated or, as this specimen was damaged during transportation, some unaccounted damage may have occurred.

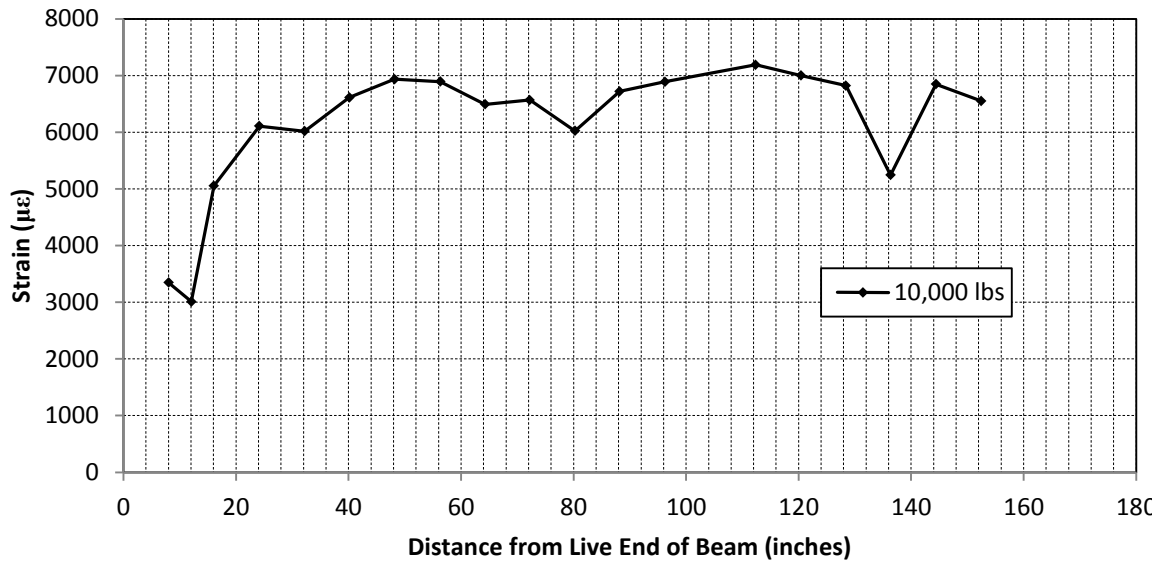


Figure 3.53 – Steel Strain vs Distance from Live End of Beam, Specimen B0.62-8

3.2.15 Specimen C0.62-8

The C0.62-6 test used the same specimen as A0.62-8A load point of 30 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 31.0 feet long with the distance from the center of the live end support to the dead end of 30.33 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.

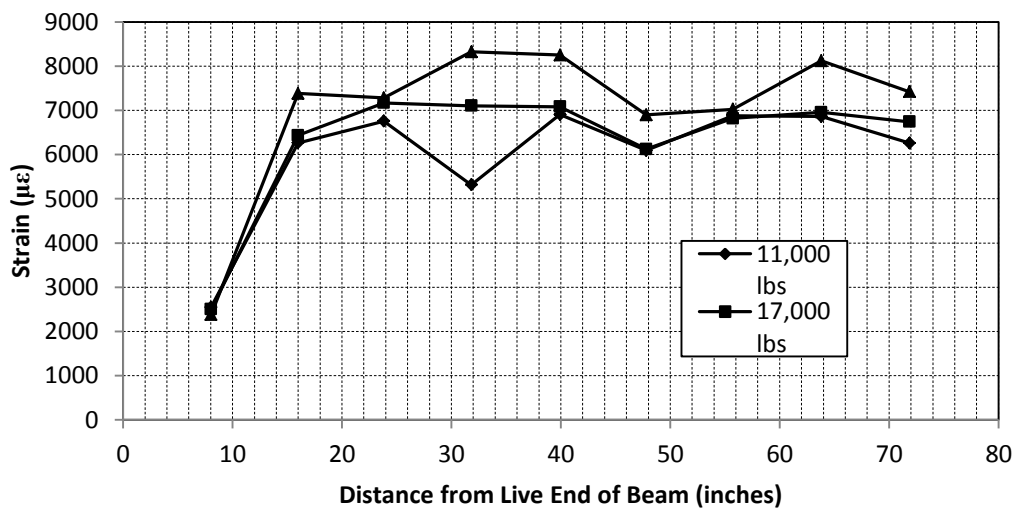


Figure 3.54 – Steel Strain vs Distance from Live End of Beam, Specimen C0.62-8

The initial cracking load was calculated to be 10.2 kips and f_{ps} moment was estimated to be 28.8 kips. From this, staged loading points of 11, 17, 23, 25, and 30 kips were selected. This specimen failed in bond slip. **Figure 3.55** shows the load versus deflection plot. Full moment capacity was not achieved in this test. The specimen had an average steel strain of 8,323 microstrain near the load point of the specimen. This specimen failed at a load of 23.9 kips. Since the strand slipped, this indicates that the development length is greater than 30 inches.

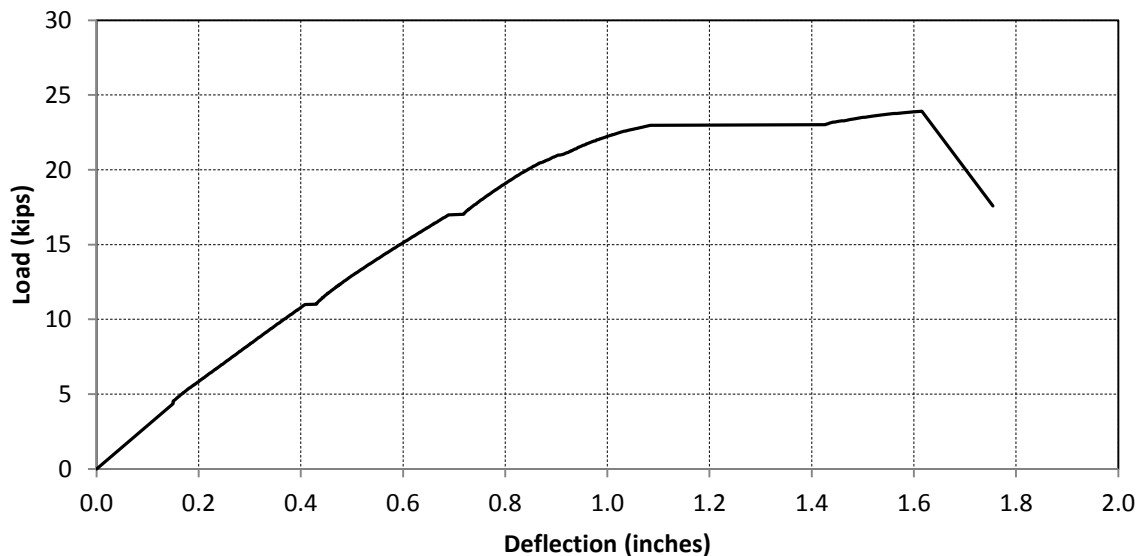


Figure 3.55 - Load versus Deflection Curve at Load Point, Specimen C0.62-8

3.2.16 Specimen A0.62-10

The A0.62-10 specimen had a 0.70-inch strand and a design concrete strength of 10,000 psi. A load point of 46 inches from the end of the specimen was selected for this test. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 50.0 feet long with the distance from the center of the live end support to the dead end of 44.60 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.



Figure 3.56 – Failure Mode of Specimen A0.62-10

The initial cracking load was calculated to be 5.5 kips and f_{ps} moment was estimated to be 15.2 kips. From this, staged loading points of 6, 8, 11, 13, and 15 kips were selected. This specimen failed in flexure and just as f_{ps} moment was achieved, the stand slipped. **Figure 3.57** shows the failure mode of the specimen. Full moment capacity was achieved in this test. The first flexural crack occurred with a load of 5 kips. This crack continued to 10.0 kips and as this loading level was attained, the beam began acting plastically. A constant deflection rate was then used to load the specimen at a rate of 0.003 in/sec until failure. An average steel strain of 19,201 microstrain near the load point of the specimen was attained. This specimen failed at a load of 12.8 kips. Some slip of the strand was observed at final loading.

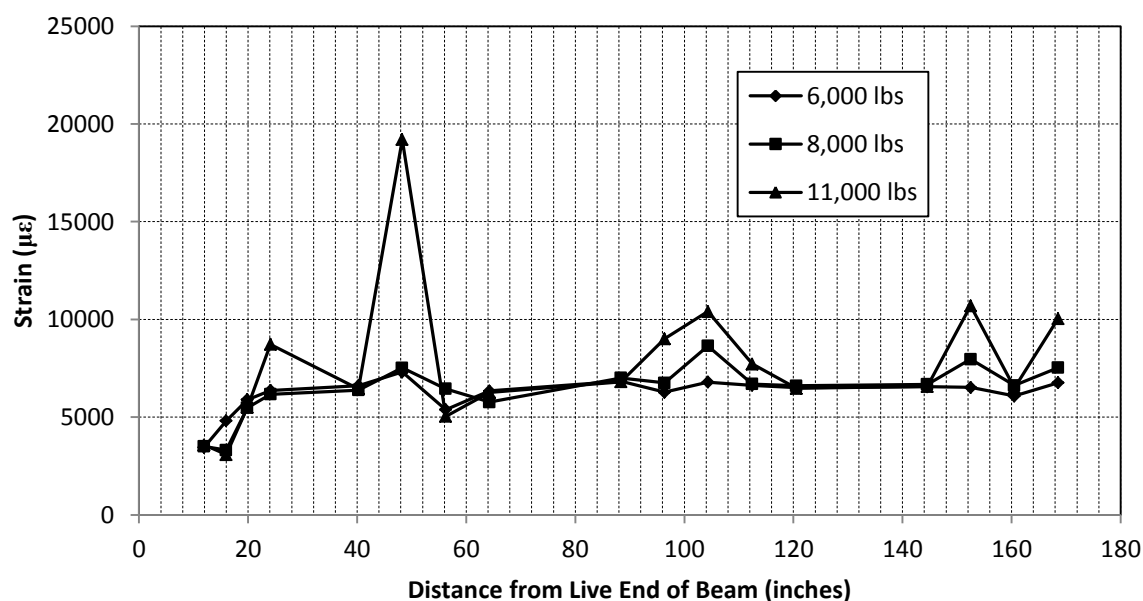


Figure 3.57 – Steel Strain versus Distance from Live End of Beam, Specimen A0.62-10

A load versus deflection curve is shown in **Figure 3.58**. As the steel reached its elastic limit, continued deflection occurred until the specimen broke as a result of a flexural failure. The concrete at the top of the specimen crushed and then the specimen failed. Since the strand slipped at the final loading just as f_{ps} moment was attained, this indicates the development length was approximately 46 inches. This test was repeated with specimen B0.62-10 at a shorter beam span, but the same load point. This was used as an attempt to demonstrate repeatability of the testing procedure.

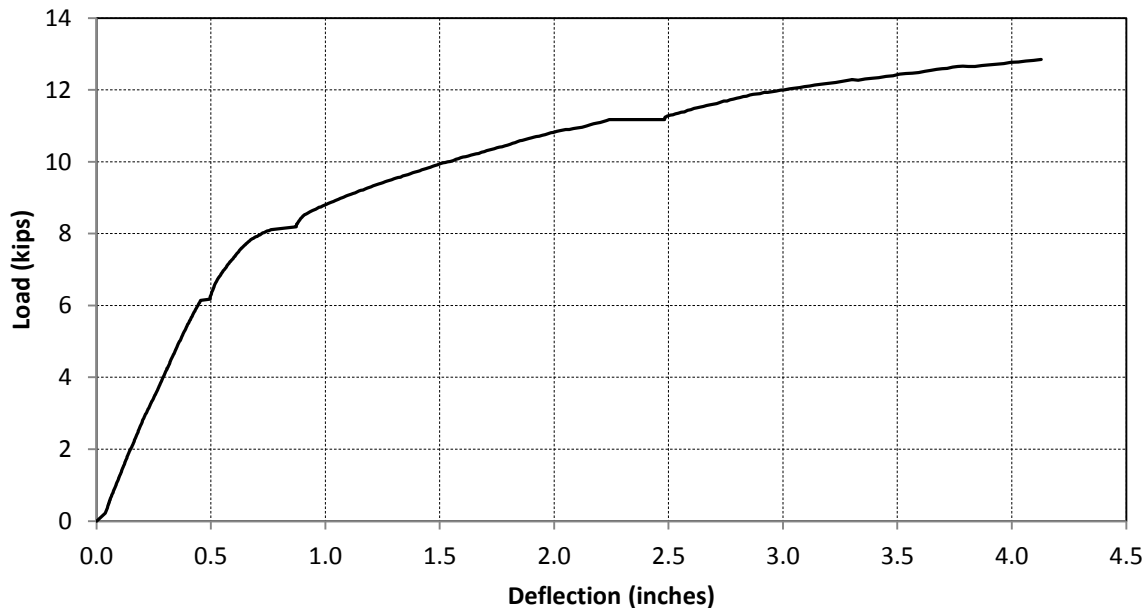


Figure 3.58 - Load versus Deflection at Load Point, Specimen A0.62-10

3.2.17 Specimen B0.62-10

The B0.62-10 specimen had 0.62-inch strand and a design concrete strength of 10,000 psi. A load point of 46 inches from the end of the specimen was selected for this test to repeat the results of specimen A0.62-10. The size of the neoprene pad was six inches at the live end and 10 inches at the dead end. The specimen was 25.0 feet long with the distance from the center of the live end support to the dead end of 24.33 feet. String potentiometers were placed at quarter points along the specimen and one was affixed to the strand.



Figure 3.59 – Failure Mode of Specimen B0.62-10

The initial cracking load was calculated to be 5.5 kips and f_{ps} moment was estimated to be 15.2 kips. From this, staged loading points of 6, 9, 12, 14.0, and 15 kips were selected. This specimen failed in flexure just as the specimen was going into a plastic behavior. This was similar to specimen A0.62-10. A final loading of 14.0 kips was reached. **Figure 3.60** shows the load versus deflection curve.

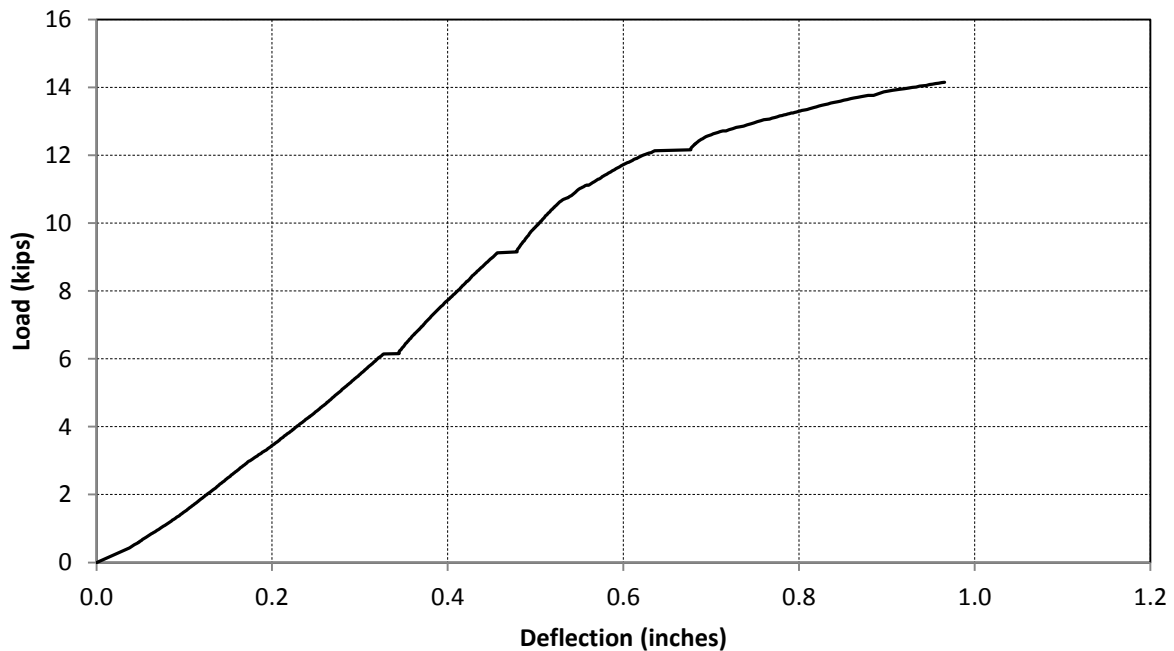


Figure 3.60 - Load versus Deflection at Load Point, Specimen B0.62-10

The first flexural crack occurred with a load of 5 kips. This crack continued to 14.0 kips and as this loading level was attained the beam began acting plastically. An average steel strain of 7,440 was attained at a load of 12.0 kips. Strand slip was observed at final loading.

This specimen behaved as the previous test, confirming the repeatability of the procedure. The development length of the 0.62-inch strand, with a 10,000 psi design strength mix, was slightly over 46 inches.

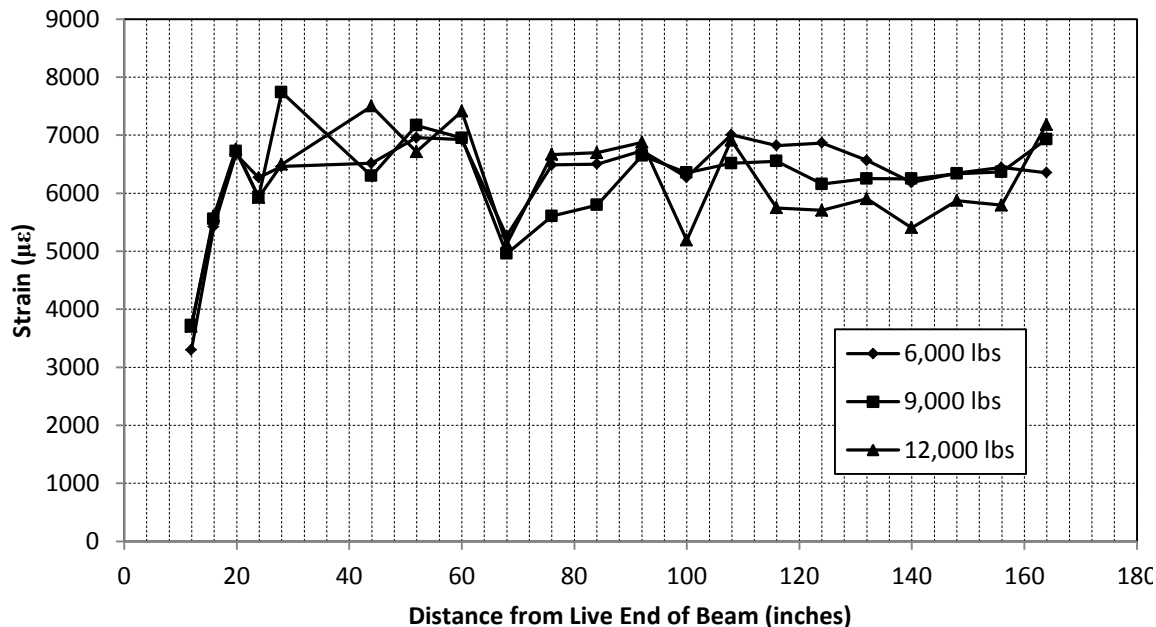


Figure 3.61 – Steel Strain vs Distance from Live End of Beam, Specimen B0.62-10

3.3 Summary of Small-Scale Test Results

Table 3.5 summarizes the results, and the data are plotted in **Figure 3.62**. A few items were obvious. The development length was found to be inversely proportional to the size of the strand and the concrete strength. The development length was considerably larger for normal strength concrete, 6,000 psi design strength or less, and has less of an effect as the concrete strength is increased.

Table 3.5 – Summary of Small-Scale Testing Results

Load Point (in)	Compressive Strength (psi)	Specimen	Failure Type	Predicted Failure (k)	Actual Failure Load (k)
30	6,366	B0.70-6	Slip	24.9	17.9
37.5	6,366	E0.70-6	Slip	30.8	26.1
43	6,366	D0.70-6	Flexure	25.4	21.3
48	6,366	C0.70-6	Flexure	18.8	24.0
98	6,366	A0.70-6	Flexure	6.9	7.5
24	8,395	B0.70-8	Slip	28.7	27.9
34	8,395	A0.70-8	Flexure	20.4	23.3
30	12,712	A0.70-10	Flexure/Slip	21.2	20.7
33	12,712	B0.70-10	Flexure	24.6	24.7
72	6,366	A0.62-6	Slip	14.1	8.7
125	6,366	C0.62-6	Flexure	11.8	12.9
	6,366	B0.62-6 - Broken Specimen			
30	8,395	C0.62-8	Flexure	14.1	15.9
40	8,395	B0.62-8	Slip	24.6	12.9
48	8,395	A0.62-8	Slip	28.8	23.9
46	12,712	A0.62-10	Flexure/Slip	15.2	12.8
46	12,712	B0.62-10	Slip	15.2	14.0

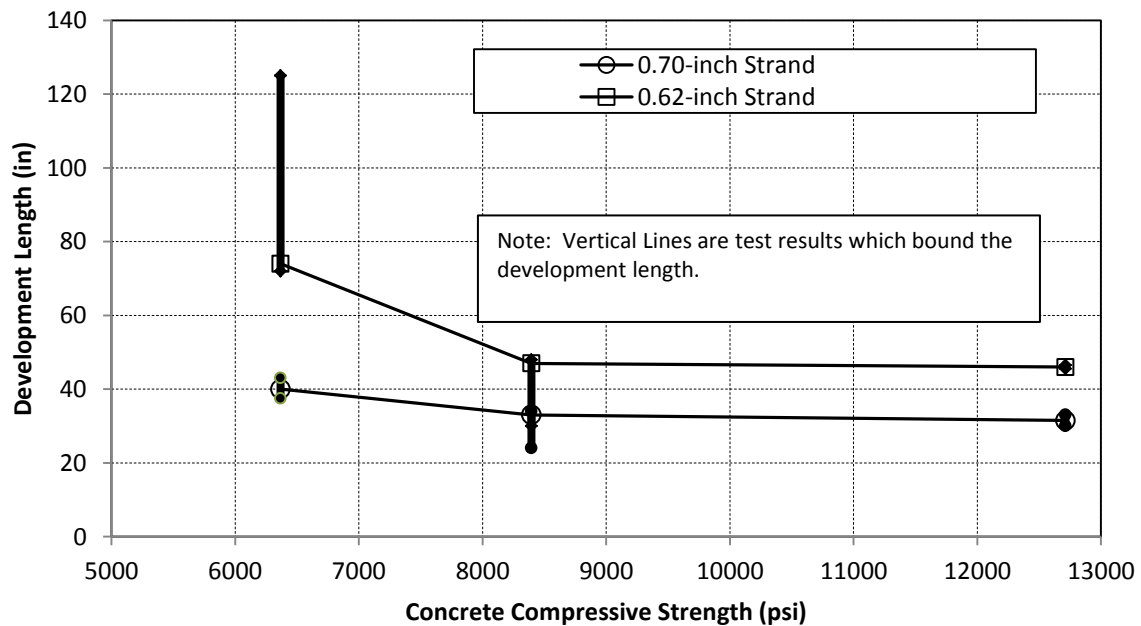


Figure 3.62 – Plot of Testing Results, Strand Development Length versus Concrete Compressive Strength

The ACI model used to predict the strength of the specimen worked well for the specimens with larger diameter strand and high strength concrete. From a design standpoint, this is important to predict the future response using this strand.

Figure 3.62 is a plot that summarizes the testing data. The horizontal lines show the bounding tests conducted using the different concrete compressive strengths. The left hand point was the test result for specimen A0.70-10 where a slip failure occurred at a distance of 30 inches. This demonstrates that the development length of this strand with this concrete was larger than 30 inches. The point on the right is for specimen B0.70-10. The test demonstrated that development length for 0.70-inch strand was less than 33 inches as a flexure failure resulted with this load point.

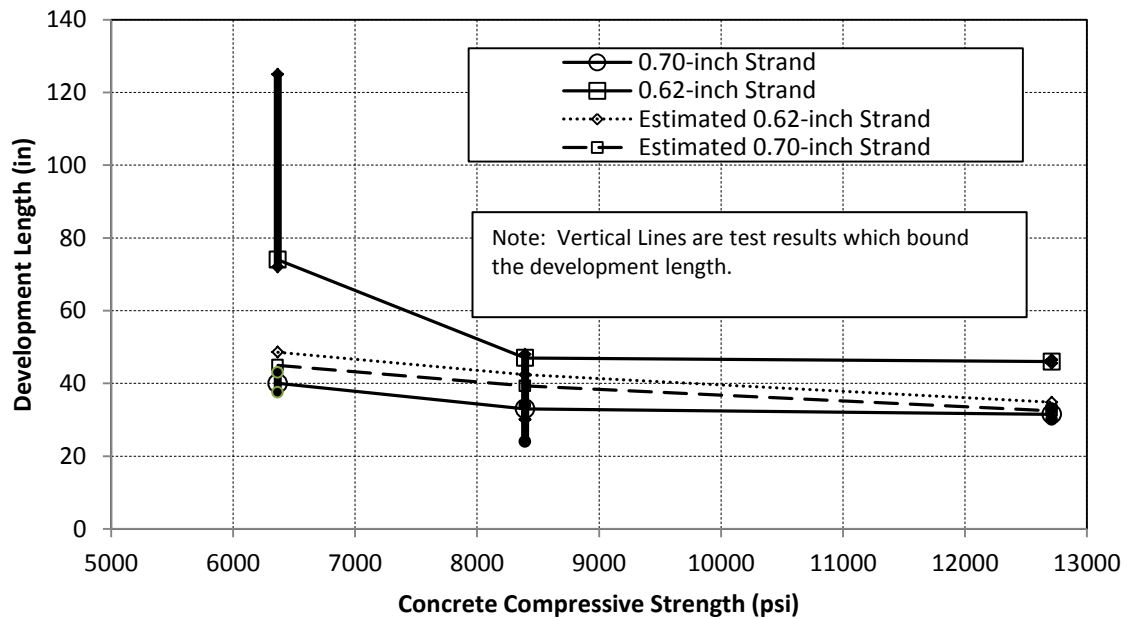
Similarly, the other horizontal lines were drawn using the results of the other tests that bounded the development length. The point on the line selected for the 0.70- and 0.62-inch curves was based upon the comparison of the predicted failure load and the failure load of the slipped strand. The closure these two values, then a point closer to the slip measurement was selected. A line was then drawn between these points.

Another observation was that 6,366 psi concrete required 50 percent more development length for strand of the same size than 8,395 psi concrete. The development length did not appreciably change for concrete strengths of 12,712 and 8,395 psi. This demonstrates that for this strand, raising the concrete strength has a diminishing effect upon development length. This seems to hold true for both sized strands.

The strand size has a distinct influence upon development length. The plot shows that the smaller strand requires a longer development length. This is counterintuitive. What this does show is that the frictional resisting forces created by prestressed strand release creates greater bond than in smaller strand.

3.4 Comparison of Development Length Modeling and Test Results

Chapter 2.0 provides a basis for the mechanics of strand interaction within the concrete. The frictional force developed from the release of the strand provides the mechanism whereby transfer and development lengths for strand of any size can be estimated. **Figure 3.63** is a merging of the data from **Figure 3.62** and **Table 2.3**.



**Figure 3.63 – Strand Development Length versus Concrete Compressive Strength
Model Comparison**

The model, as presented, gives a general estimation of the development length of the strand. It tends to underestimate the development length for the 0.62-inch strand while slightly overestimating the development length for the 0.70-inch strand.

This approach does capture the lengthening of development length for the smaller strand as was shown by the small-scale testing results presented in Chapters 2 and 3, and shows the relationship between the strand development length and concrete compression strength.

4.0 FULL-SCALE TESTING

In February 2014, the load frames and testing equipment were moved from NTRC to the High Bay Area at the University of Tennessee. Two AASHTO Type I beams with 0.62-inch and 0.70-inch strand were relocated to the structural laboratory as well.

This chapter describes the work that was performed to prepare the specimens for testing. This included the building of formwork, developing hydraulic controls, programming and use of electronic measuring devices, predictions of capacities using existing models, procedures and physical measurements used in testing, and a summary of the test results for the two full-scale tests which were conducted. Findings of the entire investigation are included in **Chapter 5.0**.

Due to time limitations, only one of these beams, the one with 0.70-inch strand, was tested at the time of this report.

4.1 Casting of Composite Deck

The first task which was conducted was the strengthening of the existing bearing supports that were purchased with the load frame that the small-scale used in testing. These supports were stiffened, wide flange sections, but they did not have enough cross sectional area to support the high reactionary forces which were anticipated during the full-scale testing. As a result, additional ½-inch plates and stiffeners were welded to the supports. Once these were modified, the AASHTO Type I beam with 0.70-inch strand was placed on top of the supports.



Figure 4.1 – Close-up of Support Modifications

Originally it was thought that a two-inch neoprene pad placed between the beam and the support would provide sufficient bearing support to allow an even distribution of force from the beam to the support. After the first test, a bearing pad failure was observed at the live end of the tested

specimen. This pad was replaced using a cotton duck bearing pad capable of withstanding the high bearing pressure which occurred during testing.

With the beam set, four prebuilt metal side supports were constructed and placed two on each side of the beam and two at each end. This was done to prevent the beam from twisting too much and falling to the floor upon loading. Based upon experience from the small-scale testing, it was found that the specimens may rotate under a slight eccentric load. These supports were anchored to the strong floor and placed within $\frac{1}{2}$ inch of the specimen. They were designed to allow vertical movement of the specimen and did not allow much rotational movement. During the second test, the composite deck sheared off on one side causing the beam to rotate at failure. These supports kept the beams from overturning and proved to be sufficient for their purposes.



Figure 4.2 – Side Supports

After placement of the side supports, 2 x 4 walls were constructed and anchored to the strong floor at a distance of about three feet from the center of the Type I beam. The walls were approximately 55 inches tall and consisted of a single bottom plate and a double top plate. A Williams Threaded Bar to 1-1/4-inch course threaded adapters were used with 1-1/4-inch threaded rods to anchor the walls to the floor. Loose 2 x 4's were placed underneath the bottom plates of the wall to facilitate form removal. When removing the forms, the floor anchors could be loosened and the 2 x 4's removed. This would allow lowering of the bottom panels.

Next, a total of 14 bottom panels were constructed. These served as a form to the underside of the deck, allowed for vertical forming at the haunch, and provided a work platform. They were constructed using 23/32-inch plywood. The camber of the beam was nearly 5 inches so along

one eight-foot side a 2 x 6 was placed to cover the haunch. On the other eight-foot side, a 2 x 4 was attached. Studs were placed at every 2 feet on center in the transverse direction. A jig was created to allow for the quick construction of this formwork. The substructure was pneumatically nailed using 3 inch long, 0.120-inch framing nails. The plywood was attached using 1-1/2-inch pneumatic staples. Four sets of three holes were predrilled on the 2 x 6's. They were placed at 1-1/2, 3, and 4-1/2 inches from the top of the 23/32-inch plywood. These were there to allow the fastening of the 2 x 6 to the Type I girder using tapcon screws. The holes were on different elevations due to the camber of the beam. The panels were placed on top of the wall and leveled. The panel was braced into position and tapcons were placed at 2 feet on center—one screw for each set of three holes. The supporting wall was then plumbed. The panel was attached to the wall and the adjacent panel at every two feet using drywall screws.

Figure 4.3 presents a picture of this formwork. Following placement of the panels, end panels were constructed. These were approximately nine feet long and three feet wide. A staircase as shown in **Figure 4.4** was constructed on one end of the end panel. This allowed controlled access to the work area and provided a safe means of entry and egress to the work platform. As an additional precaution, a ledger board with studs was affixed to the 2 x 6's and supported upon the concrete floor. This was to provide a secondary means of load support at the beam side of the formwork rather than relying upon the tapcon screws for the full support.



Figure 4.3 – Construction of Formwork Support Structure Showing Beam Side Supports



Figure 4.4 – Staircase for Work Access

Figure 4.5 shows the hand rails that were constructed around the entire structure. These were built in 16-foot sections which would allow the handrails to be taken apart easily when the forms were removed. The top of the hand rails were constructed out of 2 x 6 material and the side rails and kick board were constructed out of 2 x 4 material. After their placement, 2 x 8's were used for the side forms of the composite deck. Triangular supports were made and these supports were fixed to the wooden platform using drywall screws. The formwork was then cleaned with a shop vacuum. Strain gauges were affixed to five of the shear reinforcement bars extending from the Type I Beam. These were placed one-inch from the top of the beam. Two different methods of designing shear reinforcement were used when developing these specimens. The AASTHO method was used for one side and the Strut-and-Tie method was used for the other. The AASHTO method provided for a greater amount of vertical reinforcement and this side was tested first. On the strong side, strain gauges were placed on the vertical reinforcement at 11.5, 29.75, 59.625, 81.75, and 142 inches. On the weak side these were placed at 6, 31, 63.25, 87, and 148.75 inches. After placement, the strain gauges were coated in latex caulk. When the caulk was used, a 90% survival rate of the strain gauges occurred. The wires were secured to the wooden forms and placed out of the way. The concrete was vibrated in place since a low water-to-cement ratio concrete was going to be used. The forms were oiled at the contact surface between the wood and concrete using waste hydraulic oil.



Figure 4.5 – Handrails, Composite Deck Form Boards, and Form Preparation

The deck was cast using a high-strength Type I/II mix which achieved a 7-day strength 7,451 psi, a 28-day strength of 9,575 psi, and a strength near the time of testing of 10,631 psi. A higher strength mix was used to shorten the width of the composite deck and yet still maintain a deck equivalency of a deck with a 10-foot width—which is a common deck width for a beam of this size using 4,000 psi concrete. At the time of pour, concrete cylinder specimens were prepared, and slump measurements were taken. Air entrainment additives were not used since the concrete would not be exposed to weather. **Figure 4.6** shows the measurement of concrete properties and preparing of the cylinders at the time of pour. Initially the concrete had a 4.75-inch slump, but during the pour, the slump became considerably less than that. Due to the high cement ratio in the mix, it was necessary to add a retarder to the mix to slow the curing process. Nevertheless, there were no problems associated with mix placement. Twelve cylinders were prepared for compression testing and placed into a humidity-controlled environment.



Figure 4.6 – Cylinder Preparation and Concrete Property Measurement at Placement

Figures 4.7 through 4.9 show the concrete placement and final poured deck. The whole process was completed within four hours and the forms were entirely removed the following day. Student labor was used to complete the concrete forming and placement task. The yard bucket was graciously supplied by Britton Bridge at no expense to the University.



Figure 4.7 – Concrete Deck Finishing



Figure 4.8 – Concrete Placement using a Yard Bucket



Figure 4.9 – Completed Deck Pour

Following the concrete pour and after the removal of the forms, a considerable amount of time was spent preparing a hydraulic actuator system and preparing the final testing program to evaluate the full-scale specimens. A large shear force was required to evaluate the end zone behavior using high-strength concrete and larger diameter strands.



Figure 4.10 – Series of Actuators to Test Beam Specimens

4.2 Hydraulic Control Systems

Figure 4.10 shows the hydraulics for the larger diameter strand test set-up. Four hydraulic actuators were used to place a point load of 460 kips in between the middle actuators. Each actuator was equipped with an individual load cell which allowed the recording of the force applied to the specimen. The two middle actuators were MTS actuators and were controlled by their computer automated system. The other two were older actuators that the University had used in years past. Both of the older actuators were tested and it was found that the seals were bad in one of them. The hydraulics had to be disassembled and the actuator had to be rebuilt with new seals installed.

During trial testing, an attempt was made to control one MTS actuator and one of the old actuators using the computer-controlled servo which controlled the direction, pressure and the amount of flow. This was attempted by using the ports in the servo block and tying the activation ports of the cylinder to these ports. It was found that the fluid control was not fast enough to stabilize both cylinders. The hydraulic lines were then reworked by adding tees between the hydraulic power unit and the hydraulic servo manifold (HSM). Hydraulic fluid was directed to each of the old cylinders in this manner. The fluid pressure and return lines were directed into a fluid directional valve. When activated, this valve could send hydraulic fluid into the extension or retraction port of either cylinder. The valve had a neutral position. In the neutral position, the valve allowed the fluid to be recirculated from the pressure side to the return hydraulic lines. Since the tees were placed upstream of the HSM, pressure could not be applied to the HSM and consequentially the MTS actuators. To correct this problem, manually-controlled ball valves were placed upstream of the directional control valves. This allowed the older actuators to be energized first and when the desired pressure was applied using the older actuators, the ball valves would then be closed applying full pressure to the HSM.

During testing of the system, we noticed that the older actuators, though working well, could not fully maintain their position when subjected to load nearing the capacity of the actuator. Due to this, shims were fashioned to help lock the actuators in place. The shims were pre-cut and made in various lengths so that they could be interchanged during a change in loading. The capacity of one of the older actuators was 150 kips and the other 200 kips. A stiff beam was used to tie all the actuators together. The load point was positioned so that it was at a distance whereby the moment from each side was balanced at the load point. The MTS actuators were positioned two feet each side of the load point. The MTS system was used to stabilize the fluid reduction in the older actuators. Once the staged load point was achieved and the shims in place, the displacement position of the MTS actuator was noted. The MTS system was set to displacement and the load on the MTS actuators was automatically changed to maintain this displacement. The change in MTS loading and the load upon the old actuators was recorded during the measurements taken during the staged loading points.

4.3 Instrumentation

Several different types of instrumentation were used. Strain gauges were used to measure the longitudinal strain of the vertical shear reinforcement within the haunch of the composite deck. Strain gauges were affixed to the embedded vertical reinforcement at one inch above the top of the beam.

String potentiometers were used to measure the vertical displacement of the beam. These string potentiometers were placed at each support and at the load point and at quarter points along the length of the beam. This set-up is shown in **Figure 4.11**. A jack stand was used to elevate the string potentiometer and was attached using masking tape. The string potentiometer had a spring-loaded string which when extended or retracted sent an electrical signal to the LabView data collection system. A stiff wire was used to extend the string so that it would not exceed its operational limits. This instrumentation was calibrated so that the LabView system recorded the actual deflection. The string potentiometers were placed at the supports so that the deflections within the bearing pad could be accounted for in any deflection analysis.



Figure 4.11 – String Potentiometer used in Measuring Beam during Testing

The MTS actuators had their own load and positional sensors. **Figure 4.12** presents a photograph of the MTS control screen used for the testing. The upper right section controls the hydraulic pump. The upper left section shows a real-time load and deflection screen so that the position and the load can be monitored graphically during testing.

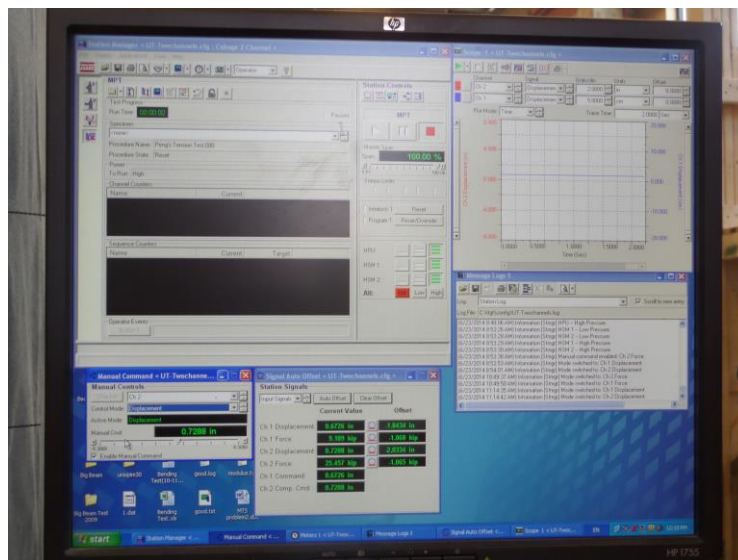


Figure 4.12 – MTS Control Screen

In the lower left was a control where the MTS actuators could be activated. They could either be controlled by deflection or load. The middle bottom section provided meters whereby the load

and deflection for each MTS actuator could be read in real time. MTS was contacted to see if the data from the MTS machine could be directly read into LabView data collection system. The MTS technical representative said that later models of their control system could do that, but our current system could not do this. The section of the screen in the lower right was the message board for this system. This recorded error and faults in the system when the pump was activated and any change in the system controls occurred.

Another string potentiometer was positioned to measure the transverse movement at the interface between the haunch and the top of the beam. **Figure 4.13** shows the positional set up for this string potentiometer. Two screws mounted the string potentiometer to the top flange of the beam. The string of the potentiometer was stretched and mounted to the composite deck. The string was positioned level at a distance parallel to the composite haunch concrete.



Figure 4.13 – String Potentiometer used to Measure Horizontal Shear at Deck Interface

Two load cells were used for the older University actuators. **Figure 4.14** shows a photograph of one of these load cells. Each of the two load cells were calibrated using a recently calibrated universal compression testing machine at the University of Tennessee. Wiring diagrams were attained from the manufacturer of the load cells and the load cells were tied into the LabView automated data collection system.



Figure 4.14 – 150 kip Actuator with Load Cell Underneath

The LabView data collection system was programmed to provide real time monitoring and was used to record data at prescribed intervals. The load cells were connected to 9237 modules using an RJ50 connector. The pin positions of these connectors were confirmed with LabView and verified during calibration. The connection data was recorded in the field notes included as **Appendix D** of this report. The strain gauges and string potentiometers were connected to an SCXI system. LabView programming was complicated by the fact that the 9237 and SCXI required different run speeds. The chassis for these systems had to be programmed pin parallel. The strain gauge and string potentiometer data had to be attained in series. To operate real time monitoring and save the data at a prescribed interval, the data for each channel had to be separated, decimated, and recombined in the programming.

Figure 4.15 presents the control screen of the programmed system. The photo on the right shows the whole screen while the one on the left is a close up of the lower right section. The front panel provides three real time monitors. The upper left one monitors the strain gauge with an activation switch and file name if the data is desired to be saved. The string potentiometer real time screen was in the upper right with an activation switch to save the information and the load cell data monitor was in the lower with and had its own activation switch. The close up picture on the right shows a numeric meter for each individual load University load cell and the addition of the two readings was given below. The actual sampling rate could be set and monitored during the test. Also in this section was a comment section. In this section, the load from the MTS system was recorded. This provided a real-time record of the total load applied by the load frame. In this manner changes in MTS load could be recorded simultaneously with the acquisition of data being collected by the LabView system.

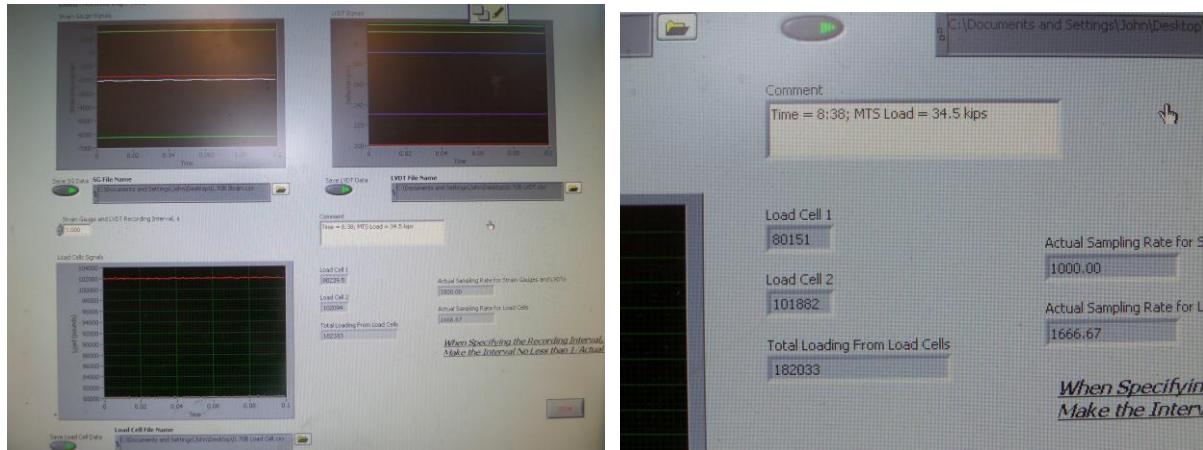


Figure 4.15 – Programmed LabView Front Panel

This LabView program was designed to handle five strain gauges, eight string potentiometers, and two load cells. Options for strain gauge shunt calibration, zeroing out the readings for load cells, and strain gauges, strain gauge factors, units calibration of load cells, sampling rate, and recording rate were included in this programming. The LabView system was found to be difficult to program. Even though there were a number of examples to go by and their technical support was fantastic, the programming was not very intuitive and took a long time to conduct. Also, the programming tended to be difficult when numerous chassis were used.

Strand slip was monitored in two different ways for the two full scale tests. For the first test, spring-loaded DC-DC Linear Variable Displacement Transducers (LVDTs) were clamped to the strand using hose clamps and cable connectors that were available at the lab. The manufacturer was contacted and a wiring diagram was obtained for use. The manufacturer said that they could be used in two different modes, a variable voltage mode or a constant voltage mode of 5.0 volts. **Figure 4.16** shows the set-up used for this test. A piece of Plexiglas was epoxied to the concrete to provide a surface to measure against. The LVDTs were a direct read system where the voltage registered nearly a 1:1 correlation to the actual displacement. The effective range of the LVDT was from 2 inches to -2 inches.



Figure 4.16 – LVDTs used for Strand Slip Monitoring

There were four wires for each LVDT, one for excitation positive, another excitation negative, a third for output positive, and a fourth for output negative. A 0 to 30 volt power supply was used to excite the system. A wiring board which had been used for previous testing was found and affixed to the end of the beam. Excitation positive wires were connected to the lower left, negative to the lower right and these were connected to the power supply. The output wiring, both positive and negative, was positioned at the top of the panel. Positional measurements were taken at each staged load point using a voltmeter. The position of the LVDT was set initially at approximately +1 inch. The LVDTs were numbered with 1 being at the top left and progressing from left to right with 12 being the lower right strand. The data at staged loading is included with the data sheets for the full-scale tests in **Appendix E**.

The sensitivity of the voltage measurement was changed from what is shown in the photograph. With the change in sensitivity, the LVDTs could be used to measure the deflection to three decimal places. An attempt was made to wire these measurements into the LabView data collection system. Upon investigation it was found that this could be done with the equipment that the University had, but it would further complicate the programming already in place. Ideally, a different module, which the University did not have, could be used to simplify the process. The decision was made to use staged measurements rather than spending an inordinate amount of time trying to reprogram the system.

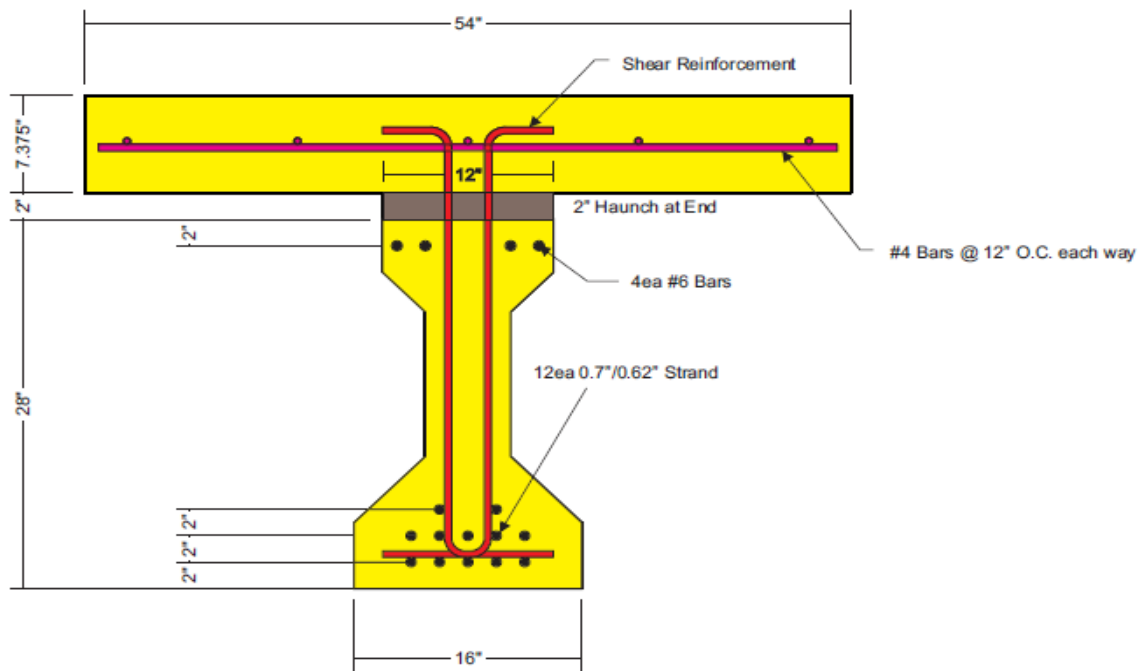


Figure 4.17 – Strand Slip Measurement for Second Full-Scale Test

For the second test, the strands were hot bent, turned up and cast within a concrete block. Strain gauges were placed on three of the strands at one inch from the end of the beam. Again due to the complexities of reprogramming the LabView system, a Vishay P3 device was used to monitor strand strain within the cast block.

4.4 Capacity Predictions using Existing Models

This section describes the detailed calculations of predictive models that were used to predict the behavior of the full-scale specimens under load. Methods examined include Compression Field Analysis, Modified Compression Theory, AASHTO General Procedures, AASHTO Simplified Procedure, and Response 2000. The predicted responses are summarized at the end of this section and were used to set load staging points and monitor the behavior of the beam while being tested.



Composite Beam Elevation

	Beam Properties without Deck	Composite Section Properties
Area (inches ²)	276	566
h (inches)	28	37,375
I (inches ⁴)	22,750	72,850
y_b (inches)	12.59	21.58
y_t (inches)	15.41	15.80
S_b (inches ³)	1,807	3,376
S_t (inches ³)	1,476	4,612
Weight (lb/ft)	288	590
f'_{ci} (ksi)	10.6	-----
f'_c (ksi)	14.1	8.5
Strand: 0.70-inch diameter, low relaxation Area of Strand, $A_p = 0.294$ inches ² Ultimate Strength of Strand, f_{ps} or $f_{pu} = 270$ ksi Yield Strength of Strand, $f_{py} = 0.9 f_{pu} = 243$ ksi Strand Stress before Transfer, $f_{pi} = 0.75 f_{pu} = 182$ ksi Strand Stress after Relaxation, $f_{se} = 152$ ksi		

Figure 4.18 – Beam Properties for Full-Scale Test

4.4.1 Compression Field Analysis

The Type I AASHTO girder with 0.7-inch strand is shown in **Figure 4.18**. The objective of the testing procedure was to subject the specimen to a severe shear and moment loading and monitor the specimen's response. This objective was accomplished by using a point load of 400 kips—the capacities of the four actuators—as near the end of the beam as possible. The first step was to determine the moment capacity, the associated shear, and distance from the end of the beam using a 400 kips loading. The beam was set up with an 8-inch bearing pad support at one end and an overhang on the other. The distance between the centers of the supports was 45 feet.

Using a strength analysis, a moment capacity of 2,290 k-ft was calculated with the compression block depth, a , being 3.597 inches and the depth to the neutral axis, c , being 5.533 inches. If a 400 kip loading was used to achieve this moment, then the distance required to achieve the service limit moment was 6.776 feet or 81.316 inches (0.143L of 45-foot Span Length).

An idealized stress/strain curve for concrete where the concrete cracks at a strain of 1000 $\mu\epsilon$, and the maximum strain in the concrete of 3000 $\mu\epsilon$ was used in this analysis. From this, the average longitudinal strain at the centroid of the composite section was calculated and the shear strain was approximated using the equilibrium and compatibility equations presented by Collins and Mitchell (1991).

Longitudinal strains were calculated for the loading progression of the beam. First, the strain due to the release of the strand was calculated in the following manner:

$$\begin{aligned}\epsilon_{P/A} &= \frac{f_{se}A_{ps}}{A_{Beam}E_{Conc Beam}} & (4.1) \\ &= \frac{-152 \text{ ksi} \left(0.294 \frac{\text{in}^2}{\text{strand}} \times 12 \text{ strands}\right)}{276 \text{ in}^2 (6,768 \text{ ksi})} \\ &= -0.000287 \text{ in/in (Compression)}\end{aligned}$$

Next, the strain created by the eccentricity of the strand to the centroid of the beam to the top and bottom extreme fiber is as follows:

$$\begin{aligned}\epsilon_{Pe/S_{top}} &= \frac{f_{se}A_{ps}e}{S_{top}E_{Conc Beam}} & (4.2) \\ &= \frac{152 \text{ ksi} \left(0.294 \frac{\text{in}^2}{\text{strand}} \times 12 \text{ strands}\right) (12.59 \text{ in} - 3.5 \text{ in})}{1,476 \text{ in}^3 \times 6,768 \text{ ksi}} \\ &= 0.000488 \text{ in/in (Tension)}\end{aligned}$$

$$\begin{aligned}
\varepsilon_{Pe/S_{bottom}} &= \frac{f_{se} A_{ps} e}{S_{bottom} E_{Conc Beam}} \\
&= \frac{-152 \text{ ksi} \left(0.294 \frac{\text{in}^2}{\text{strand}} \times 12 \text{ strands} \right) (12.59 \text{ in} - 3.5 \text{ in})}{1,807 \text{ in}^3 \times 6,768 \text{ ksi}} \\
&= -0.000399 \text{ in/in (Compression)}
\end{aligned} \tag{4.3}$$

Also, the strain created by the dead weight—at the critical section—of the beam and composite deck at the top and bottom extreme fiber is:

$$\begin{aligned}
\varepsilon_{M_{DL} top} &= \frac{M_{DL}}{S_{top} E_{Conc Beam at Release}} \\
&= \frac{(-48.52 \text{ k} - ft) 12 \text{ in/ft}}{1,476 \text{ in}^3 \times 5,861 \text{ ksi}} \\
&= -0.0000673 \text{ in/in (Compression)}
\end{aligned} \tag{4.4}$$

$$\begin{aligned}
\varepsilon_{M_{DL} bottom} &= \frac{M_{DL}}{S_{bottom} E_{Conc Beam at Release}} \\
&= \frac{(48.52 \text{ k} - ft) 12 \text{ in/ft}}{1,807 \text{ in}^3 \times 5,861 \text{ ksi}} \\
&= 0.0000550 \text{ in/in (Tension)}
\end{aligned} \tag{4.5}$$

Rather than a simplified compression block, a limit analysis was used to better approximate the longitudinal strain at the centroid of the strand pattern. Several equations of equilibrium must be satisfied simultaneously to satisfy a solution. These equations are as follows:

$$\begin{aligned}
C_{TOTAL} &= C_1 + C_2 \\
C_1 &= (x) (b_{f_{eff}}) (0.85 f'_c) = 201.9 x \\
C_2 &= \frac{1}{2} (y) (b_{f_{eff}}) (0.85 f'_c) = 100.97 y
\end{aligned} \tag{4.6}$$

The distance (L) from the top of the composite deck to the resultant C_{TOTAL} occurs at the following location:

$$L = \frac{[C_1(x/2) + C_2(y/3 + x)]}{[C_1 + C_2]} \tag{4.7}$$

$$T = M_{LL}/d_v = (2,290 \text{ k} - ft)x 12 \frac{\text{in}}{\text{ft}} / (33.875 \text{ in} - L)$$

$$= (201.9x + 100.97y)$$

$$y = 0.003(33.875 \text{ in} - x) / (\varepsilon_{ps} + 0.003)$$

Using Excel, a trial and error solution was found. The results are as follows:

$$\begin{aligned} x &= 2.88 \text{ inches} \\ y &= 2.84 \text{ inches} \\ c &= 5.72 \text{ inches} \\ L &= 2.228 \text{ inches} \\ C_1 &= 582.0 \text{ kips} \\ C_2 &= 286.4 \text{ kips} \\ C_{TOTAL} &= 868.4 \text{ kips} = T \\ \varepsilon_{ps} &= 0.029778 \text{ in/in (Tension)} \\ \varepsilon_{top \text{ of the beam}} &= 0.000003 \text{ (Tension)} \\ \varepsilon_{bottom \text{ of beam}} &= 0.033480 \text{ (Tension)} \end{aligned}$$

Summing each load condition the strain at the top and bottom of the beam becomes as follows by superposition:

$$\begin{aligned} \varepsilon_{top \text{ of the beam}} &= -0.000287 + 0.000488 - 0.000067 + 0.000003 = 0.000137 \text{ (Tension)} \\ \varepsilon_{bottom \text{ of beam}} &= -0.000287 - 0.000399 - 0.000055 + 0.033480 = 0.032739 \text{ (Tension)} \end{aligned}$$

Considering the strain condition at the center of the beam alone, the longitudinal strain (ε_x) at that point is 0.016742 in/in (Tension).

The shear at a distance 6.776 feet from the end of the beam was calculated to be 355.4 kips. This creates an average shear stress which can be calculated as follows:

$$f_y = f_t = \frac{|V_u - \phi V_p|}{\phi b_v d_v} \quad (4.8)$$

$$= \frac{(355.4 \text{ kips} - 0 \text{ kips})}{(1.0 \times 6 \text{ in} \times 31.65 \text{ in})} = 1.871 \text{ ksi}$$

$$\varepsilon_t = \varepsilon_{xy} = \frac{f_y \text{ or } f_t}{E_{conc}} = \frac{1.871 \text{ ksi}}{57 \sqrt{14,100}} = 0.000277 \text{ in/in} \quad (4.9)$$

Using Compression Field Equilibrium equations in **Chapter 1.0**,

$$Y_{xy} = 2 (\varepsilon_x - \varepsilon_2) \cot \theta \quad (4.10)$$

$$0.000277 = 2 (0.016742 - \varepsilon_2) \cot \theta$$

$$\cot \theta = 2 \left[\frac{0.016742 - \varepsilon_2}{0.000277} \right] = 120.88 - 7220.22\varepsilon_2 \quad (4.11)$$

and from **Figure 1.22**,

$$\begin{aligned} \varepsilon_2 &= \frac{V}{b_w d_v E_{web}} [\tan \theta + \cot \theta] \\ &= \frac{355.4 \text{ kips}}{6 \text{ in.} \times 31.65 \text{ in} \times 6,768 \text{ ksi}} \left[\frac{1}{(120.88 - 7220.22\varepsilon_2)} + (120.88 - 7220.22\varepsilon_2) \right] \end{aligned} \quad (4.12)$$

Solving for ε_2 and θ ,

$$\varepsilon_2 = 0.022417, \text{ and } \theta = 89.3^\circ. \quad (4.13)$$

Also from equations shown in **Figure 1.22**,

$$\tan^2 \theta = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_t - \varepsilon_2} \quad (4.14)$$

Giving,

$$\varepsilon_t = 0.022416 \text{ in/in}$$

And,

$$\varepsilon_1 = \varepsilon_x + \varepsilon_t - \varepsilon_2 \rightarrow \varepsilon_1 = 0.016741 \text{ in/in} \quad (4.15)$$

Based upon this, at this point under this loading, the beam is highly influenced by the applied moment and longitudinal strain as evidenced by the steep vertical crack angle.

4.4.2 Predicting Response using Modified Compression Theory

The primary difference between the Compression Theory and the Modified Compression Theory is that the Compression Theory neglects the contribution of tensile stresses in cracked concrete and the Modified Compression Theory considers the contribution of tensile stresses. Prior to

cracking, shear is carried by diagonal tensile and compressive stress. After diagonal cracking, the tensile stresses in the concrete are substantially reduced and transferred across the cracks. The tensile stresses in the concrete vary from zero at the cracks, to a maximum at the midpoint between the cracks.

The diagonal compressive stresses push apart the flanges of the beam while the tensile stresses pull them together. Based upon testing conducted by Vecchio and Collins (1988), equations for the average strain across the cracks have been developed which relate the average strains in these zones, the bond characteristics of the reinforcement and the type of loading.

In this section, the stirrups are designed for the Type I beam using the modified compression theory. The beam was divided into segments of length $d_v \cot \theta$. Parameters were taken at the middle of each segment and applied across the entire segment. Near the support the angle θ will be approximately 0° . A conservative θ angle of 30° was used to determine the length of the first segment which was calculated to be 62.8 inches.

Summarized calculations are provided in **Table 4.1** for each beam section. Example calculations are provided for the first segment.

Step 1 – Determine the Shear and Moment at the Middle of each Segment

The reaction and moments can be determined using the following equations:

$$R_L = 400 \text{ kips} \left[\frac{(45 \text{ feet} - 6.776 \text{ feet})}{45 \text{ feet}} \right] + 0.590 \frac{\text{kips}}{\text{foot}} (56 \text{ feet}) \left[\frac{45 \text{ feet} - 27.667 \text{ feet}}{45 \text{ feet}} \right] = 352.50 \text{ kips}$$

$$V_x = \begin{cases} 352.50 \text{ kips} - 0.590 \frac{\text{kips}}{\text{foot}} x (\text{feet}) & 0.33 \text{ feet} < x < 6.78 \text{ feet} \\ -47.5 \text{ kips} - 0.590 \frac{\text{kips}}{\text{foot}} x (\text{feet}) & 6.78 \text{ feet} < x < 45 \text{ feet} \end{cases}$$

$$M_x = \begin{cases} 353.46 \text{ kips} (x \text{ feet}) - \frac{\left[(x + 0.33 \text{ feet}) \left(0.590 \frac{\text{kip}}{\text{foot}} \right)^2 \right]}{2} & 0.33 \text{ feet} < x < 6.78 \text{ feet} \\ 353.46 \text{ kips} (x \text{ feet}) - \frac{\left[(x + 4''/12) \left(0.590 \frac{\text{kip}}{\text{foot}} \right)^2 \right]}{2} - 400 \text{ kips} (x - 6.776 \text{ feet}) & 6.78 \text{ feet} < x < 45 \text{ feet} \end{cases}$$

Table 4.1 summarizes each of the calculated values for this example and the shear and moment for each section are presented.

Step 2 – Determine $d - a/2$ for each Section

From equilibrium,

$$M_x / (d - a/2) = 0.85 f'_c b_f a \quad (4.16)$$

$$a \left(d - \frac{a}{2} \right) = \frac{M_x}{[0.85 f'_c b_f]} \quad (4.17)$$

For a moment of 923.66 k-ft and $d = 33.8756$ inches,

$$a = d - \sqrt{d^2 - \frac{2M_x}{0.85 f'_c b_f}} \quad (4.18)$$

$a = 1.282$ inches which is less than the deck height of 7.375 inches

$$d - a/2 = 33.875 - 1.282/2 = 33.234 \text{ inches} \quad (4.19)$$

The term “ jd ” then becomes $d - a/2$ but not less than $0.9 d$ or $0.72 h$, which are 30.488 inches and 26.91 inches. The rest of the sections are calculated and summarized in **Table 4.1**.

Step 3 – Design the Stirrups

For each section determine v/f'_c :

$$v/f'_c = \left(\frac{V_u}{\phi} - V_p \right) / (b_w jd f'_c) \quad (4.20)$$

For the first section, this is:

$$\begin{aligned} v/f'_c &= (350.96 \text{ kips} / 0.85 - 0 \text{ kips}) / (6 \text{ inches} \times 33.23 \text{ inches} \times 14.1 \text{ ksi}) \\ &= 0.147 \end{aligned} \quad (4.21)$$

With this v/f'_c and assuming $\varepsilon_x \approx 0$, a θ of 25° is used as an initial estimate for angle determination. The value of ε_x then can be estimated using the following equation:

$$\varepsilon_x = [M_u/jd + 0.5N_u + 0.5V_u \cot \theta - A_{ps} f_{se}] / [E_s A_s + E_p A_{ps}] \quad (4.22)$$

$$\begin{aligned} &= [923.67 \times 12 / 33.23 + 0 + 0.5(350.96) \cot(25^\circ) - 12 \times 0.294 \times 152] / [28,500(12) 0.294] \\ &= 0.001726 \text{ in/in} \end{aligned}$$

From **AASHTO LRFD Table S5.8.3.4.2-1**, θ is 34° by interpolation. Revising the estimate for θ to 33° , ε_x becomes 0.000671 in/in. Again, from **AASHTO LRFD Table S5.8.3.4.2-1**, $\theta = 33^\circ$ and $\beta = 2.08$.

The concrete capacity can be expressed as:

$$V_c = \beta \sqrt{f'_c} b_w j d \quad (4.23)$$

$$= 2.08 (14,100)^{1/2} (6'')(33.23'') / 1000 \text{ pounds/kip} = 49.2 \text{ kips}$$

Thus,

$$V_s = V_u / \phi - V_c - V_p \quad (4.24)$$

$$= 350.96 - 49.2 - 0 = 301.7 \text{ kips}$$

Also,

$$V_s = A_v f_y j d \cot \theta / s \quad (4.25)$$

Assuming double legged #4 stirrups,

$$s \leq 0.4 \text{ in}^2 (60 \text{ ksi}) (33.23 \text{ in}) \cot(33) / 301.7 \text{ k} = 4.07'' \quad \text{Use } 4''$$

Minimum shear reinforcement spacing is as follows:

$$s \leq A_v f_y / 50 b_w = 0.4 \text{ in}^2 (60,000 \text{ psi}) / 50 (6'') = 80'' \quad (4.26)$$

Other minimum spacing requirements:

$$s \leq 0.75 h = 0.75(37.375'') = 28.0'' \quad (4.27)$$

Step 4 – Double Check the Longitudinal Reinforcement

The longitudinal reinforcement must be capable of resisting the force generated by the moment and pressure applied to the horizontal reinforcement by the concrete compression strut within the beam's web. This is given by the following equation:

$$V_{s \text{ Actual}} = 0.4 \text{ inches}^2 \times 60 \text{ ksi} \times 33.23 \text{ inches} \cot(33) / 4 \text{ inches} = 307.0 \text{ kips} \quad (4.28)$$

$$A_{ps} f_{ps} = M_u / (\phi j d) + (V_u / \phi - 0.5 V_s - V_p) \cot \theta$$

$$= 923.67 \text{ k-ft} \times 12 / 33.23 + (350.96 \text{ k} - 0.5 \times 307.0 \text{ k}) \cot(33)$$

$$f_{ps} = 637.6 \text{ k} / (12 \times 0.294 \text{ in}^2) = 180.7 \text{ ksi}$$

Check to see if this stress can be developed at this section. Using AASHTO's equations, the transfer length is equal to $60 d_b$, which is 42 inches and the development length is:

$$l_d \geq \kappa [f_{ps} - 2/3 f_{pe}] d_b \quad (\text{AASHTO Equation 5.11.4.2-1}) \quad (4.29)$$

where, κ is an escalation factor of 1.6 specified by AASHTO

$$= 1.6 [270 \text{ ksi} - 2/3 (152 \text{ ksi})] (0.5'') = 135''$$

This section is at 31.4 inches + 4 inches = 35.4 inches from the end of the beam to the center of the support, or 35.4 inches < transfer length as predicted by AASHTO. As a result, a bond failure is predicted using this equation. Based upon small-scale testing by Jiang, Ma, and Cabage (2013), the transfer length for this size strand will likely occur at a distance of approximately 20 inches and the development length at approximately 28 inches. If the small-scale tests are predictive of multiple strand performance, then the strand should be fully developed at this section, and bond slip failure should not occur under this loading provided the end zone of the beam is not cracked.

Table 4.1 - Summary Data for each Beam Section

No.	x, ft	V_u , kips	M_u , kip-ft	$d - a/2$, in	jd , in	$v/f'c$	θ	ϵ_x , in/in	θ	V_c , kips	s, in	f_{ps} , ksi
1	2.62	351.0	924	33.2	33.2	0.24360	33	0.00067103	2.08	49.2	4	181.9
2	6.78	51.5	2386	32.2	32.2	0.03693	49.7	0.00373825	1.22	28.0	28	261.9
3	7.19	51.7	2366	32.2	32.2	0.03709	49.3	0.00366357	1.23	28.2	28	259.8
4	11.75	54.4	2138	32.4	32.4	0.03882	45.8	0.00281706	1.44	33.2	28	236.9
5	16.32	57.1	1902	32.5	32.5	0.04052	42.8	0.00195414	1.74	40.3	28	213.9
6	20.89	59.8	1660	32.7	32.7	0.04220	38.6	0.00109612	2.25	52.4	28	192.5
7	25.46	62.5	1410	32.9	32.9	0.04386	32.8	0.00026514	3.29	77.1	28	176.5
8	30.03	65.2	1152	33.1	33.1	0.04549	28*	- 0.00056436	5.24	123.5	28	168.8
9	34.60	67.9	888	33.3	33.3	0.04710	28*	- 0.00151187	5.24	124.2	28	142.0
10	39.16	70.6	616	33.5	33.5	0.04869	28*	- 0.00247456	5.24	124.9	28	114.8
11	43.73	73.3	337	33.6	33.6	0.05026	28*	- 0.00345191	5.24	125.6	28	87.1
12	45.00	74.1	258	33.7	33.7	0.05069	28*	0.00372561	5.24	125.8	28	79.4

*The 28-inch spacing is a minimum spacing requirement.

4.4.3 Determine Shear Design Using AASHTO General Procedure

The shear reinforcement is designed for the same sections as the Modified Compression Theory. Only one section is presented and the rest are summarized in **Table 4.2**.

Step 1 - Determine Crack Spacing Parameter, s_{xe}

$$s_{xe} = s_x [1.38 / (a_g + 0.63)] \quad (4.30)$$

where,

s_x = the lesser of d_v or the maximum distance between layers of crack control, where the area of the reinforcement is not less than $0.003 b_v s_x$.

Since no longitudinal reinforcement is provided, $s_x = d_v$. For the first section, $d_v = 33.2''$.

$a_g = 0.75''$ which is an assumed aggregate size

$$s_{xe} = 33.2'' [1.38 / (0.75'' + 0.63)] = 33.2''$$

Step 2 - Determine Net Longitudinal Tensile Strain in Section at Centroid of Tensile Reinforcement, ϵ_s

$$\epsilon_s = \frac{\left[\frac{|M_u|}{d_v} + 0.5 N_u + |V_u - V_p| - A_{ps} f_{po} \right]}{[E_s A_s + E_p A_{ps}]} \quad (4.31)$$

$$= 0.000176 \text{ in/in}$$

$$M_u = 924 \text{ k-ft}$$

$$d_v = 33.234''$$

$$N_u = 0.0 - \text{No Axial Load}$$

$$V_u = 351.0 \text{ k}$$

$$V_p = 0 \text{ k}$$

$$A_{ps} = 12 \text{ strands} \times 0.294 \text{ in}^2/\text{strand} = 3.528 \text{ in}^2$$

$$f_{po} = 0.7 f_{pu} = 0.7 \times 270 \text{ ksi} = 189 \text{ ksi}$$

$$E_s = 29,000 \text{ ksi}; \quad A_s = 0 \text{ in}^2$$

$$E_p = 28,500 \text{ ksi};$$

Step 3 - Determine θ and β

$$\theta = 29 + 3500 \epsilon_s = 29.62^\circ \quad (\text{AASHTO Equation 5.8.3.4.2-3}) \quad (4.32)$$

$$\beta = [4.8 / (1 + 750 \epsilon_s)] \quad (\text{AASHTO Equation 5.8.3.4.2-1}) \quad (4.33)$$

Assumes Minimal Amount of Shear Reinforcement

$$\beta = 4.24$$

Step 4 - Determine V_c

$$V_c = 0.0316 \beta \sqrt{f'c} b_v d_v = 0.0316 (4.24) (14.1)^{1/2} (6'') (33.2'') = 100.3 \text{ k} \quad (4.34)$$

Step 5 - Determine V_s Required

$$\Phi V_{s \text{ Required}} = V_u - \Phi V_c - \Phi V_p = V_u / \Phi - V_c - V_p \quad (4.35)$$

Note: For this analysis $\phi = 1.0$

$$= 351.0 \text{ kips} - 100.3 \text{ kips} - 0 \text{ kips} = 250.7 \text{ kips}$$

Step 6 - Determine Transverse Shear Size and Spacing using Double #4 Stirrups

$$V_s = [A_v f_y d_v \cot \theta] / s \quad (4.36)$$

$$250.7 \text{ k} = [(2 \times 0.2 \text{ in}^2) (60 \text{ ksi}) (33.2'') \cot(29.63)] / s$$

$$s = 5.60'' \text{ use } 5'' \quad V_s @ 5'' = 280.2 \text{ kips}$$

Step 7 - Determine Minimum Steel Requirement

$$A_v \geq 0.0316 \sqrt{f'c} (b_v s / f_y) \quad (\text{AASHTO Equation 5.8.2.4-1}) \quad (4.37)$$

$$= 0.0316 (14.1)^{1/2} (6'') (18'') / (60 \text{ ksi})$$

$$= 0.213 \text{ in}^2 < 0.4 \text{ in}^2 \text{ ----- OK}$$

Table 4.2 – Summary Data for each Beam Section

No.	x, ft	V_u , kips	M_u , kip-ft	$d - a/2$, in	jd , in	s_{xe}	ϵ_x , in/in	ϑ	θ	V_o , kips	s, in	f_{ps} , ksi
1	2.62	351.0	924	33.2	33.2	33.23387	0.000176	29.62	4.24	100.3	5.60	207.1
2	6.78	51.5	2386	32.2	32.2	32.16356	0.002735	38.57	1.57	36.0	28.00*	267.9
3	7.19	51.7	2366	32.2	32.2	32.17879	0.002659	38.31	1.60	36.7	28.00*	266.0
4	11.75	54.4	2138	32.4	32.4	32.35051	0.001797	35.29	2.04	47.1	28.00*	245.1
5	16.32	57.1	1902	32.5	32.5	32.52572	0.000917	32.21	2.84	65.8	28.00*	226.6
6	20.89	59.8	1660	32.7	32.7	32.70430	0.000020	29.07	4.73	110.1	28.00*	215.9
7	25.46	62.5	1410	32.9	32.9	32.88613	0.000000	29.00	4.80	112.4	28.00*	190.5
8	30.03	65.2	1152	33.1	33.1	33.07111	0.000000	29.00	4.80	113.0	28.00*	164.1
9	34.60	67.9	888	33.3	33.3	33.25912	0.000000	29.00	4.80	113.7	28.00*	137.2
10	39.16	70.6	616	33.5	33.5	33.45007	0.000000	29.00	4.80	114.3	28.00*	109.9
11	43.73	73.3	337	33.6	33.6	33.64385	0.000000	29.00	4.80	115.0	28.00*	82.2
12	45.00	74.1	258	33.7	33.7	33.69811	0.000000	29.00	4.80	115.2	28.00*	74.5

*The minimum shear spacing requirement of 28 inches calculated in Section 4.3.2 applies to this procedure as well.

4.3.4 Determine Shear Design Using AASHTO Simplified Procedure

The shear reinforcement is designed for the same sections as the modified compression theory. Only one section is presented and the rest are summarized in **Table 4.3**.

Step 1 - Determine V_{ci}

$$V_{ci} = 0.02 \sqrt{f'_c} b_v d_v + V_d + V_i \quad M_{cre}/M_{max} \geq 0.06 \sqrt{f'_c} b_v d_v \quad (4.38)$$

$$\begin{aligned} V_d &= \text{Shear Force at Section due to Unfactored Dead Loads} \\ &= 12.7 \text{ k} \end{aligned}$$

$$\begin{aligned} V_i &= \text{Factored Shear Force at Section due to Externally-applied} \\ &\quad \text{Loads Occurring Simultaneously with } M_{max} \text{ (} \phi = 1.0 \text{)} \\ &= V_u - V_d \text{ (without factor)} = 350.1 \text{ k} - 12.7 \text{ k} = 337.4 \text{ k} \end{aligned}$$

$$M_{cre} = \text{Moment Causing Flexural Cracking at Section due to}$$

$$\begin{aligned}
& \text{Externally-applied Loads} \\
& = [6 \sqrt{f'c} + f_{pe} - f_d] S_{bc} \\
& = [712 \text{ psi} + 4,639 \text{ psi} - 221 \text{ psi}] (3,376 \text{ in}^3 / 12,000) = 1,443 \text{ k-ft}
\end{aligned} \tag{4.39}$$

$$\begin{aligned}
f_{pe} &= P_{se}/A + P_{se} e / S_b \\
&= (12 \text{ strands} \times 0.294 \text{ in}^2 \times 152 \text{ ksi}) / 276 \text{ in}^2 \\
&\quad + 536 \text{ k} (9.09'') / 1,807 \text{ in}^4 = 4.639 \text{ ksi}
\end{aligned} \tag{4.40}$$

$$f_d = M_{DL} / S_b = 33.3 \text{ k-ft} \times 12 / 1,807 \text{ in}^4 \times 1,000 = 221 \text{ psi} \tag{4.41}$$

$$\begin{aligned}
M_{max} &= \text{Maximum Factored Moment at Section due to} \\
&\quad \text{Externally-applied Loads} \\
&= M_u - M_d = 924.0 - 33.3 = 890.7 \text{ k-ft}
\end{aligned}$$

$$\begin{aligned}
V_{ci} &= 0.02 \sqrt{f'c} b_v d_v + V_d + V_i M_{cre}/M_{max} \geq 0.06 \sqrt{f'c} b_v d_v \\
&= 0.02 (14.1)^{1/2} (6'') (33.2'') + 12.7 \text{ k} + 338.3 \times 1,443 \text{ k-ft} / 890.7 \text{ k-ft} \\
&= 576 \text{ k} \geq 0.06 (14.1)^{1/2} (6'') (33.2'') = 44.9 \text{ k}
\end{aligned} \tag{4.42}$$

Use 576 k

Step 2 - Determine V_{cw}

$$V_{cw} = (0.06 \sqrt{f'c} + 0.30 f_{pc}) b_v d_v + V_p \tag{4.43}$$

f_{pc} = Compressive Stress in Concrete (after allowances for all losses) at centroid of cross section resisting externally applied loads. This stress is due to both prestress and moments resisted by precast members acting alone.

$$\begin{aligned}
f_{pc} &= P_{se}/A - P_{se} e (y_{bc} - y_b)/I + M_D (y_{bc} - y_b) / I \\
&= 536 \text{ k} / 276 \text{ in}^2 - 536 \text{ k} (9.09 \text{ in}) (21.58 - 12.59) \text{ in} / 72,850 \text{ in}^4 \\
&\quad + 33.2 \text{ k-ft} \times 12 \text{ in/ft} (21.58 - 12.59) \text{ in} / 72,850 \text{ in}^4 \\
&= 1.390 \text{ ksi}
\end{aligned} \tag{4.44}$$

$$\begin{aligned}
V_{cw} &= (0.06 \sqrt{f'c} + 0.30 f_{pc}) b_v d_v + V_p \\
&= [0.06 (14.1)^{1/2} + 0.30 (1.390 \text{ ksi})] (6'') (33.2'') + 0 \text{ k} = 128 \text{ k}
\end{aligned} \tag{4.45}$$

For Shear Capacity use $128 \text{ k} < 576 \text{ k}$

Step 3 - Determine V_s / s

$$V_s = [A_v f_y d_v \cot \theta] / s \quad \text{Assumes } \alpha = 90^\circ \tag{4.46}$$

where $V_{ci} < V_{cw}$

$$\cot \theta = 1.0$$

where $V_{ci} > V_{cw}$

$$\cot \theta = 1.0 + 3 [f_{pe} / \sqrt{f'_c}] \leq 1.8 \quad (5.8.3.4.3-4) \quad (4.47)$$

$$= 1.0 + 3 [4.639 \text{ ksi} / (14.1)^{1/2}] = 4.71 \leq 1.8$$

Use 1.8

For Double #4 Stirrups,

$$s = [A_v f_y d_v \cot \theta] / (V_u \text{ Required} / \Phi - V_c) \quad (4.48)$$

$$= [0.4 \text{ in}^2 (60 \text{ ksi}) (33.2'') (1.8)] / [(351.0 \text{ k}) / 1.0 - 128 \text{ k}] = 6.44''$$

Use 6''

Table 4.3 – Summary Data for each Beam Section

No.	x, ft	V_u , kips	M_u , kip-ft	$d - a/2$, in	jd , in	M_d , k-ft	f_d , psi	M_{cre} , k-ft	V_{ci} , k	f_{pc} , ksi	V_{cw} , k	$\cot \theta$	s, in
1	2.62	351.0	924	33.2	33.2	33.2	221	1443	576	1.390	128.08	1.8	6.44
2	6.78	51.5	2386	32.2	32.2	86.1	571	1345	50	1.468	12.79	1.8	28.00*
3	7.19	51.7	2366	32.2	32.2	91.3	606	1335	51	1.476	12.63	1.8	28.00*
4	11.75	54.4	2138	32.4	32.4	149.3	991	1227	53	1.562	10.91	1.8	28.00*
5	16.32	57.1	1902	32.5	32.5	207.3	1377	1118	57	1.648	28.00	1.8	28.00*
6	20.89	59.8	1660	32.7	32.7	265.3	1762	1010	62	1.734	28.00	1.8	28.00*
7	25.46	62.5	1410	32.9	32.9	323.3	2147	901	69	1.820	28.00	1.8	28.00*
8	30.03	65.2	1152	33.1	33.1	333.3	62	1488	123	1.834	28.00	1.8	28.00*
9	34.60	67.9	888	33.3	33.3	298.4	-497	1645	182	1.783	28.00	1.8	28.00*
10	39.16	70.6	616	33.5	33.5	263.5	-1056	1802	324	1.731	28.00	1.8	28.00*
11	43.73	73.3	337	33.6	33.6	228.6	-1615	1960	1122	1.679	28.00	1.8	28.00*
12	45.00	74.1	258	33.7	33.7	218.9	-1770	2003	3142	1.665	28.00	1.8	28.00*

*The minimum shear spacing requirement of 28 inches calculated in Section 4.3.2 applies to this procedure as well.

4.4.5 Analysis of Composite Beam using Response 2000

The Response 2000 program was developed using Modified Compression Theory by Bentz and Collins (2001). User reinforcement parameters can be included, such as 0.7-inch and 0.62-inch strand, by modifying the rebar.dat file. This was done to include these strand sizes and higher strengths into the program.

The model uses an incremental loading approach coupled with a sectional analysis approach to estimate shear and moment capacity, crack spacing and width, load capacity, tensile and compression stress/strain, as well as other parameters. An analysis was conducted for four different vertical shear reinforcement spacing. The vertical reinforcement spacing used was 4, 6, 8, and 28 inches. A moment to shear ratio was included in the loading parameter of the program. In this case, the maximum design moment was paired with the maximum shear which gives a ratio of moment to shear of 6.776:1.0.

Figure 4.19 presents a graphical summary of the parameters used for this analysis.

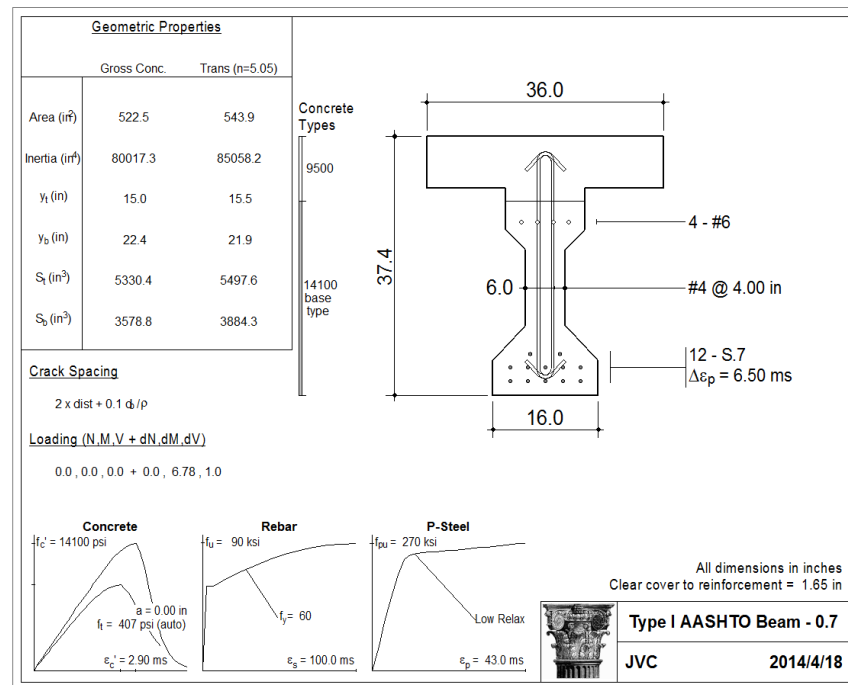


Figure 4.19 – Example of Material and Section Inputs into Response 2000, 4-inch Transverse Shear Spacing

Figure 4.20 shows the shear control graphs for the four different transverse reinforcement levels. Response 2000 predicts a different maximum shear capacity and a different mode of failure for the beam for the four different spacings. For the 4-inch and 6-inch spacing, a relatively high shear capacity is given, 315.8 and 289.9 kips respectively. The beam with these reinforcement levels are predicted to have a ductile failure. The capacity level is reduced for the 6-inch when

compared to the 4-inch spacing, but the behavior is similar and is what would be expected for a ductile failure.

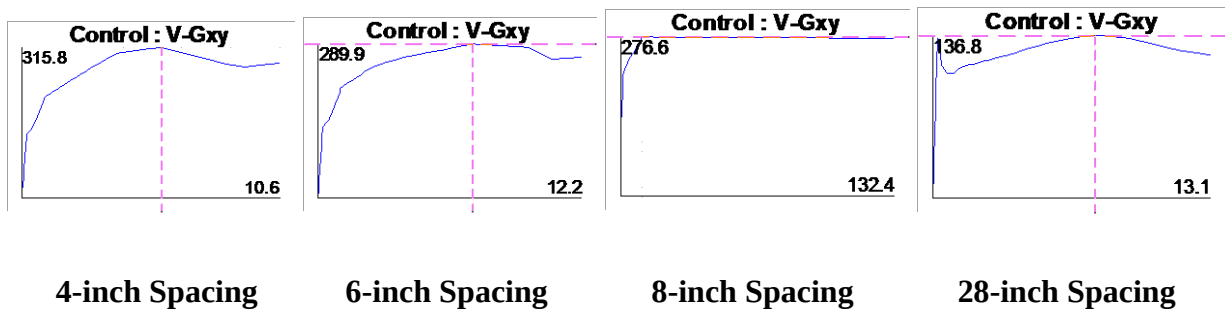


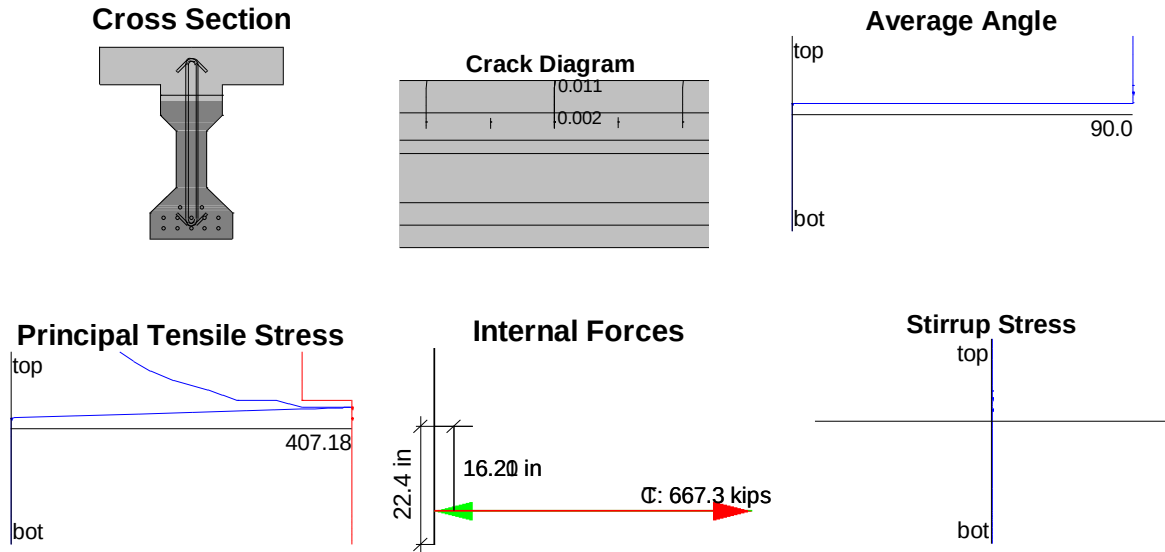
Figure 4.20 – Shear Control Curve for Varying Reinforcement Spacing

Response 2000 predicts a different type failure mode for the 8-inch and 28-inch spacing under the prescribed loading condition. With the 8-inch spacing, the steel quickly goes to yield and to strain hardening. For the 28-inch spacing, the capacity is diminished to about one-third of the 4-inch spacing capacity. Some strain hardening occurs within the transverse reinforcement, but the specimen will fail prematurely because of a lack of sufficient reinforcement under the given load.

Figure 4.21 shows a progression for the failure of the beam with transverse reinforcement at 4-inch spacing. Six different graphs are presented that show the various predicted parameters at each loading stage.

Figure 4.21 – Predicted Type I Beam Response to Progressive Loading

Stage 1 - No Loading, $V = 0$ k, $M = 0$ k-ft



Stage 2 – At Initial Cracking, $V = 134$ k, $M = 911$ k-ft

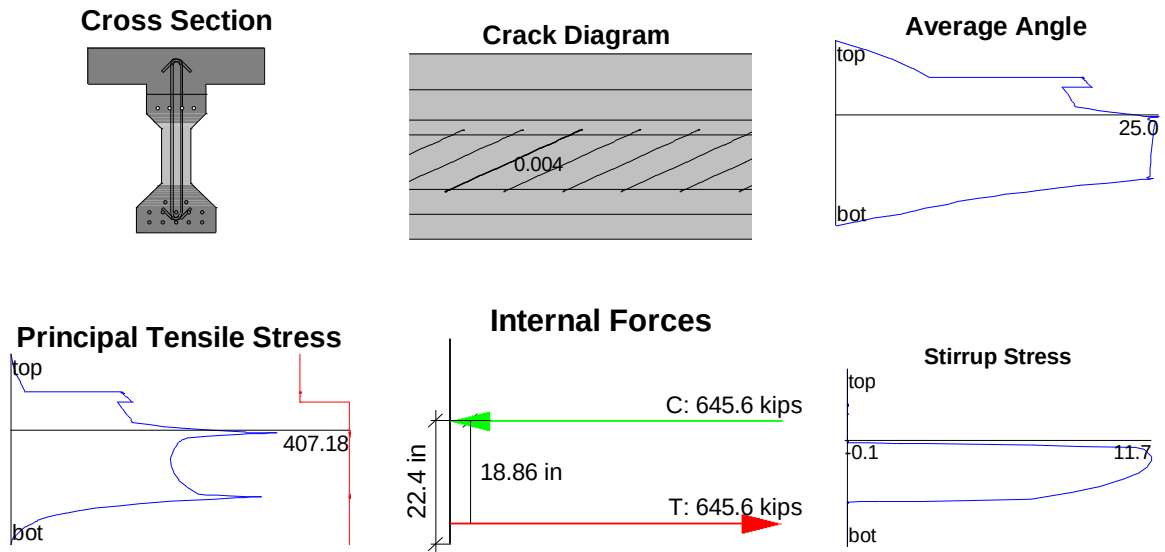
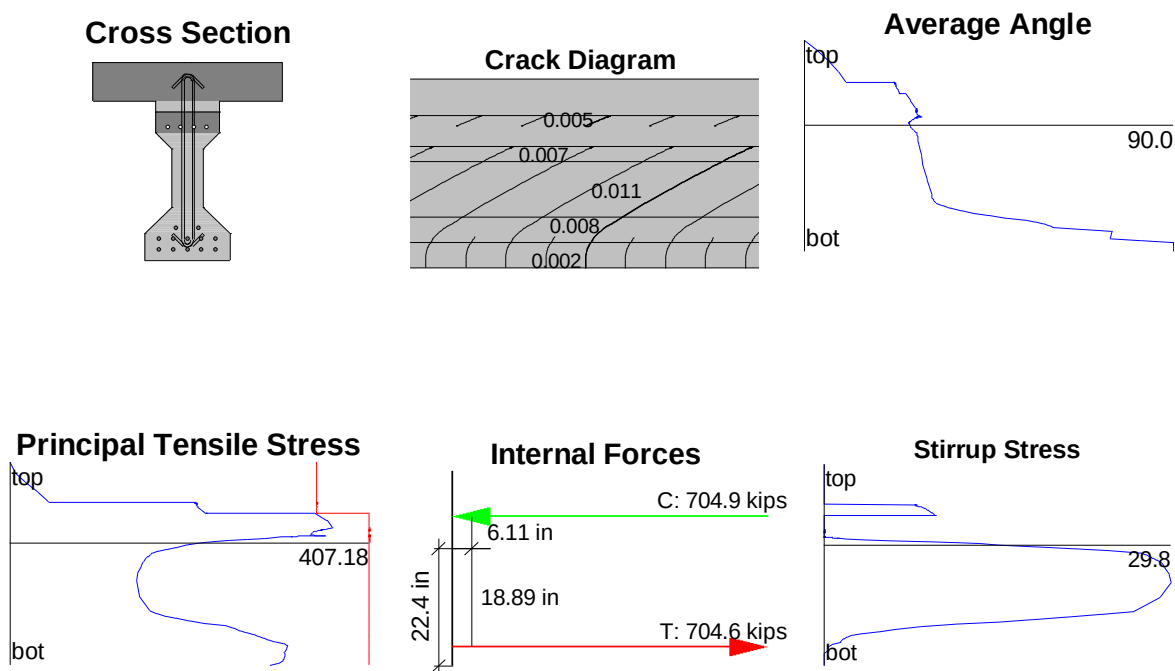


Figure 4.21 continued

Stage 3 – At Initial Yield, $V = 213.2$ kips, $M = 1468$ k-ft



Stage 4 – At Maximum Shear Capacity, $V = 316$ k, $M = 2141$ k-ft

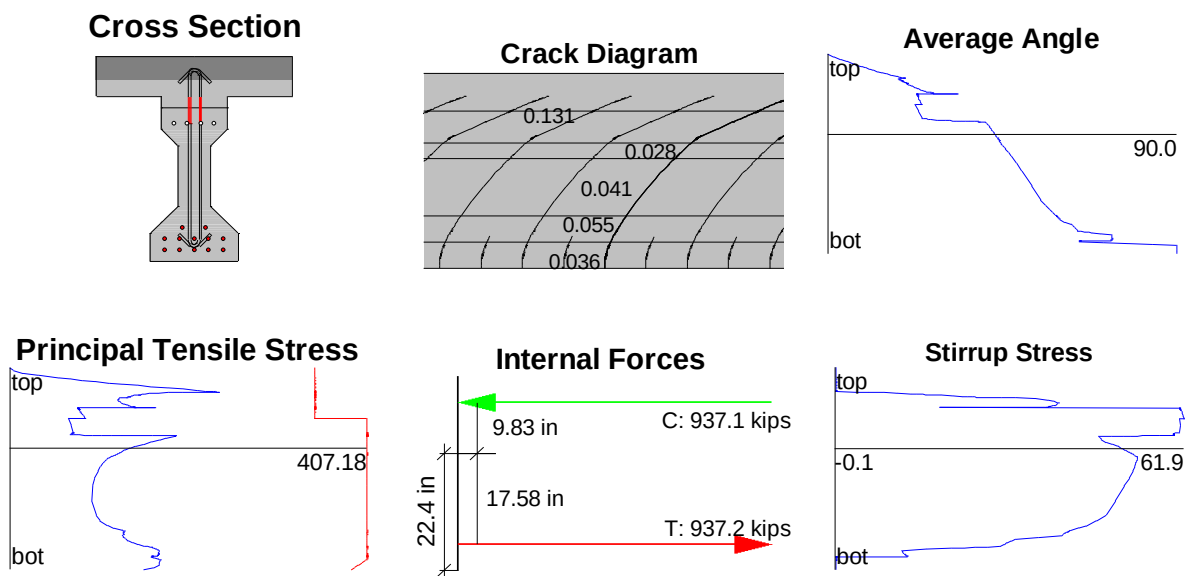


Figure 4.21 continued

Stage 5 – At Failure, $V = 283 \text{ k}$, $M = 1920 \text{ k-ft}$

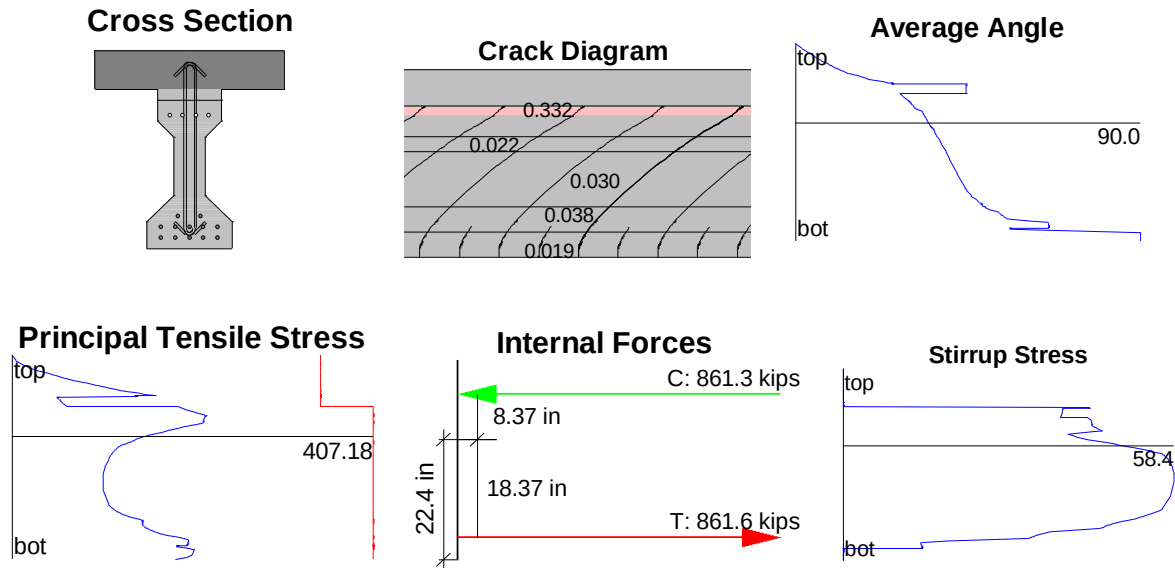


Figure 4.21 continued

For the concrete, the lighter gray color represents an area of tensile stress while the darker gray indicates compression. At zero load, the concrete beam is in compression and the deck is shown to be in tension. As the loading increases at Stage 2, the deck and flanges of the section remain in compression, but a tensile force is predicted to develop in the web of the section near the centroid of the section. At Stage 3, the moment has a stronger influence over the behavior of the section and the tensile zone extends to the extreme bottom fiber of the section. Note is that a compression zone remains in the vicinity of the longitudinal mild steel reinforcement in the top flange of the section at this Stage. By Stage 4, maximum shear capacity, the compression zone lies entirely within the deck and a longitudinal tensile stress exists in the lower section of the deck, haunch, and throughout the beam section as well.

Stress in the reinforcement of the section is illustrated by the changing of the color of the reinforcement. At Stage 1, the reinforcement is represented by hollow circles for strand and mild longitudinal reinforcement. As the stress condition changes, the reinforcement color changes to a dark red when the reinforcement is yielded and a bright red when the strain of the reinforcement is within the strain hardening zone. Also, note that gray is also used in the reinforcement to indicate compression. At Stage 3, the concrete is in tension and the strand is at yield. By Stage 4, the strand has yielded in tension and the transverse reinforcement has yielded and is in the state of strain hardening. Once permanent plastic deformation has occurred, Stage 5, the capacity of the section is reduced in shear and in moment and with the reduced capacity, stress is alleviated from the section.

Predicted crack diagrams are also shown for each loading stage. At Stage 1, the Response 2000 program predicts that tensile cracks will develop at the top of the deck. While tensile cracks were observed in the top of the beam, this cracking will not extend to the top of a deck poured subsequent to the beam itself. The program applies the initial strain in the strand of $6,500 \mu\epsilon$ assigned to the strand to the entire section upon release. This could distort the strain distribution throughout the section and could cause some error in the predictions.

At Stage 2, Response 2000 predicts that web cracking will begin to occur. The cracks are predicted to be 0.004 inches in width and based upon the crack angle graph expected to be at 23° from the axis of longitudinal reinforcement. The principal tensile stress graph indicates that the principal tensile stress has not exceeded the tensile capacity of the beam concrete. This seems to be in error as the principal tensile stress must be exceeded before cracking can initiate. For this loading, it does seem reasonable that the compression field may be at 23° , but additional loading, beyond the rupture strength of the concrete, is required for cracking to ensue.

At Stage 3, the compression field at the bottom fiber of the section becomes highly influenced by the added moment onto the section. Response 2000 predicts that the crack angle is nearly vertical in the bottom flange. The compression field becomes steeper and the Response 2000 estimates the width of the cracks to be nearly 0.011 inches.

At Stage 4, Response 2000 predicts that the compression field and associated cracking within the web of the section to be between 45 and 60 degrees. Crack widths by this analysis are expected to be 0.055 inches at the bottom flange and web interface and 0.131 inches at the interface between the haunch and beam section.

At Stage 5, final failure was predicted and shown by the yielding of the concrete at the intersection of the top flange and the web of the section. This is shown by the concrete at this section being colored pink.

Two other sets of graphs are presented in **Figure 4.21**. These are an internal forces plot which show the compression and tensile force acting at the compression block and the centroid of the strand, respectively, and a plot predicting the stirrup stress with depth. Using the Compression Block model, a moment capacity of 2,290 k-ft was approximated for this section under the loading considered. This is compared with a capacity of 2,141 k-ft predicted by Response 2000. This is a 6.5% difference for the two predictions which is relatively close.

Response 2000 predicts that at Stage 4, a maximum stirrup stress of 61.9 ksi will be attained at maximum load capacity. This stress is predicted to occur from the interface of the deck and beam through the top flange of the beam portion of the section.

4.5 Testing Procedures and Physical Measurements

Several different physical measurements were desired during the testing of this beam. First a grid of DEMEC points was placed within the web of the beam to measure the average strain. The points were glued in an orthogonal pattern with two-inch spacing both horizontally and vertically. Six rows and 45 columns were placed on the web. At three places in the grid, the rows had to be shifted upward to maintain a level horizontal grid due to the camber in the beam. **Figure 4.22** shows the grid and the use of a dial gage measuring device to measure between the points. As the measurements were taken, they were called out and recorded on an Excel spreadsheet. The individual recording the measurements would then call out his response and the person doing the measurements would verify the recorded measurement's accuracy. Also, the individual recording the measurements would compare the measurement to the previous stage's measurement and check both for reasonableness. Unreasonable data was rechecked before proceeding.



Figure 4.22 – DEMEC Gauge Pattern and Measurement between Points

When the DEMEC points were set, a prepositioned gauge bar was used to place the points. The bar allowed for an accurate gauge measurement of 2, 4, 6, 8, and 10 inches. The horizontal gauge could measure to the nearest 0.00001 inch at each of these gauge points. A four-inch gauge was used to measure the average strain between points for this grid. The aggregate size the beam was 1-3/8 inches, so this gauge provided for approximately four times the aggregate size.

A Gilson dial gauge was used to measure the horizontal measurements. This gauge was calibrated so that a measurement of 0.25000 inches represented four inches while measuring the grid. The gauge operated so that recorded measurements with a larger number were actually shorter than the gauge distance. For instance a gauge measurement of 0.35000 inches would be 3.9 inches. A more compact dial gauge had to be used for the vertical measurements. A Starrett gauge was found in the instrumentation room and was used to measure the grid vertically. The Starrett gauge measured in SI units. The gauge was set so that a measurement of 0.6000 was 400 millimeters—approximately four inches. The higher the number on this gauge, the shorter the distance. Similarly, a gauge measurement of 0.5000 millimeters meant that the distance was 399.1 millimeters. Both gauges were used simultaneously to shorten the time required to measure the grid at staged loading points.

A second set of DEMEC points were positioned along the centroid of the strand pattern at the bottom flange of the beam. These were to measure the average strain the longitudinal strand reinforcement. An eight-inch gauge length was used to measure these points. The line of DEMEC points was placed at four-inch centers.

Physical measurements of beam deflection were also taken at the support, load point, and quarter spans of the beam. These were performed to validate the calibration of string potentiometers used for the same purpose. Measurements were taken and recorded for the DC voltage LVDTs and the strain gauges affixed to the extended strands which were encased in concrete at the end of the beam. These recorded measurements are included in **Appendix E** of this dissertation.

Loading points were selected for various stages of beam behavior. Response 2000 predicted that the beam would initially crack at a shear of 110 kips, corresponding to a loading of 130 kips, yield at a shear of 213 kips, corresponding to a load of 252 kips, maximum capacity at a shear of 316 kips, corresponding to a loading of 374 kips, and a failure after exceeding beam capacity of 283 kips, corresponding to a loading of 335 kips. Based upon this and other models, load points for the first test were selected at 110 kips, slightly before cracking, 185 kips, slightly before yield, 235 kips, slightly after yield, 300 kips, within the plastic range of the beam, 350 kips, near failure, and at failure. The second test was modified slightly based upon results from the first where the points selected were at 140 kips, 230 kips, 280 kips, 350 kips, and failure.

At each load step the beam was inspected for cracking. The cracks were marked using a different color marker at each load point. **Figure 4.23** is a photograph showing marked cracks. Cracks were also marked at intermediate loading steps.

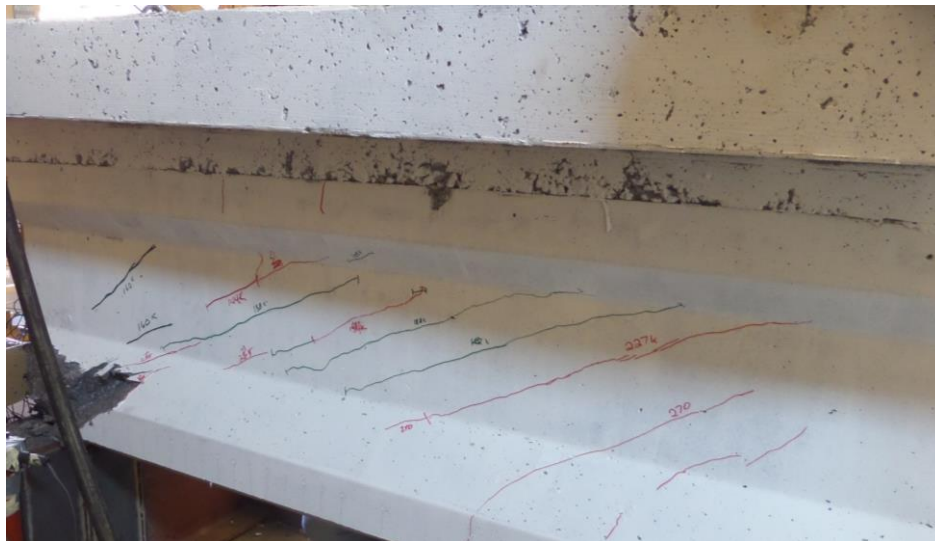


Figure 4.23 – Crack Marking at Staged Load Points

4.6 Summary of Test Results

Test 0.70A was conducted over two days due to the considerable amount of time it took to record the DEMEC grid measurements at each loading step. The first load step was 110 kips, and no cracking was observed at this load stage.

During the first load step, the Plexiglas plate that was attached at the end of the beam moved due to excessive deformation of the neoprene pad. While the loading was paused, the lower portion of the Plexiglas was cut off and refastened to the beam. There was no slip observed at an applied loading of 110 kips, so the movement of the Plexiglas plate should not affect the overall results.

The load was increased to 185 kips, pausing periodically to observe cracks that were forming. **Figure 4.24** is a photo of the cracks. Two web cracks formed initially at an applied load of 144 kips. One was about 36 inches from the end of the specimen and the other was about 60 inches from the end. Also, two vertical cracks were noted at about 40 inches and 60 inches from the end of the beam in the top flange.

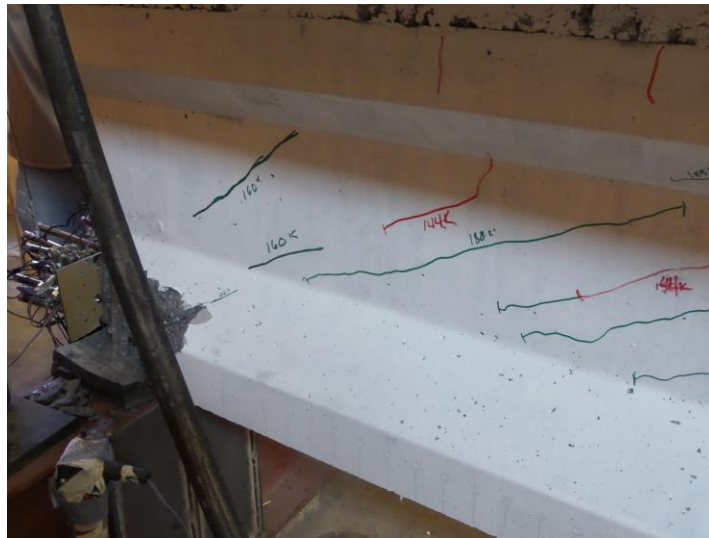


Figure 4.24 – Initial Cracks between 110 and 185 kip Load

At 185 kips, vertical cracks began to form in the flanges at the support bearing pad location. The 2-inch neoprene bearing pad was not sufficient to handle the pressure being applied to it. The pad did not allow sufficient rotation nor did it evenly distribute the load across the support structure. It is suspected that the difference in stiffness between the outer flanges and the web caused a shear force at this interface.

Figure 4.25 shows a series of photographs of the flange cracks at the end zone. **Figure 4.25 (a)** shows the unpainted side and shows the fracturing of the flange. Note that the web shear cracks extended from the web into the flange and weakened the connection between the web and flange of the beam. The flange on the painted side also fracture and spalled off. **Figure 4.25 (b)** shows the spalled flange. The crushed neoprene pad can also be seen in this photograph. The vertical height of the neoprene after testing was found to be less than $1/8^{\text{th}}$ of an inch.



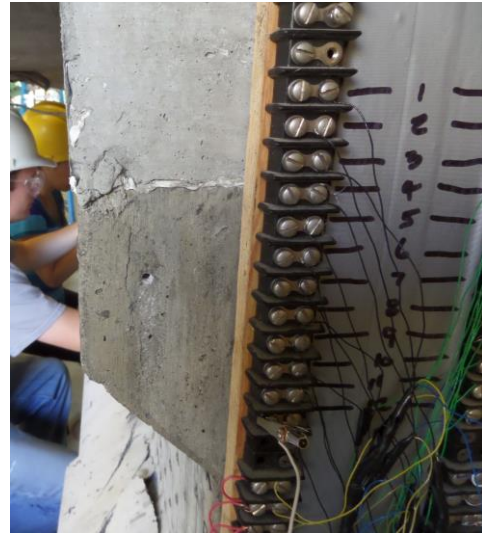
(a)



(b)



(c)



(d)

Figure 4.25 – Flange Cracks between a Load of 110 and 185 kips

Another interesting observation was the extension of the shear cracks into the bearing area of the beam. Typically, the vertical reaction forces provide sufficient compression so that there is a relatively crack free zone at the support. It is obvious from **Figure 4.25 (c)** that the web shear cracking extended into the support area. This type of disturbance into the end region of the strand adversely affects the bond of the strand to the concrete.

Figure 4.25 (d) shows the formation of a flange crack at the top flange of the beam. This vertical crack developed at 185 kips. The top flange did not spall off. One major difference in this flange and the bottom flange was that web cracking did not extend into the top flange in this area. This flange cracking shows that the end zone is susceptible to shear cracking between the web and flange interface.

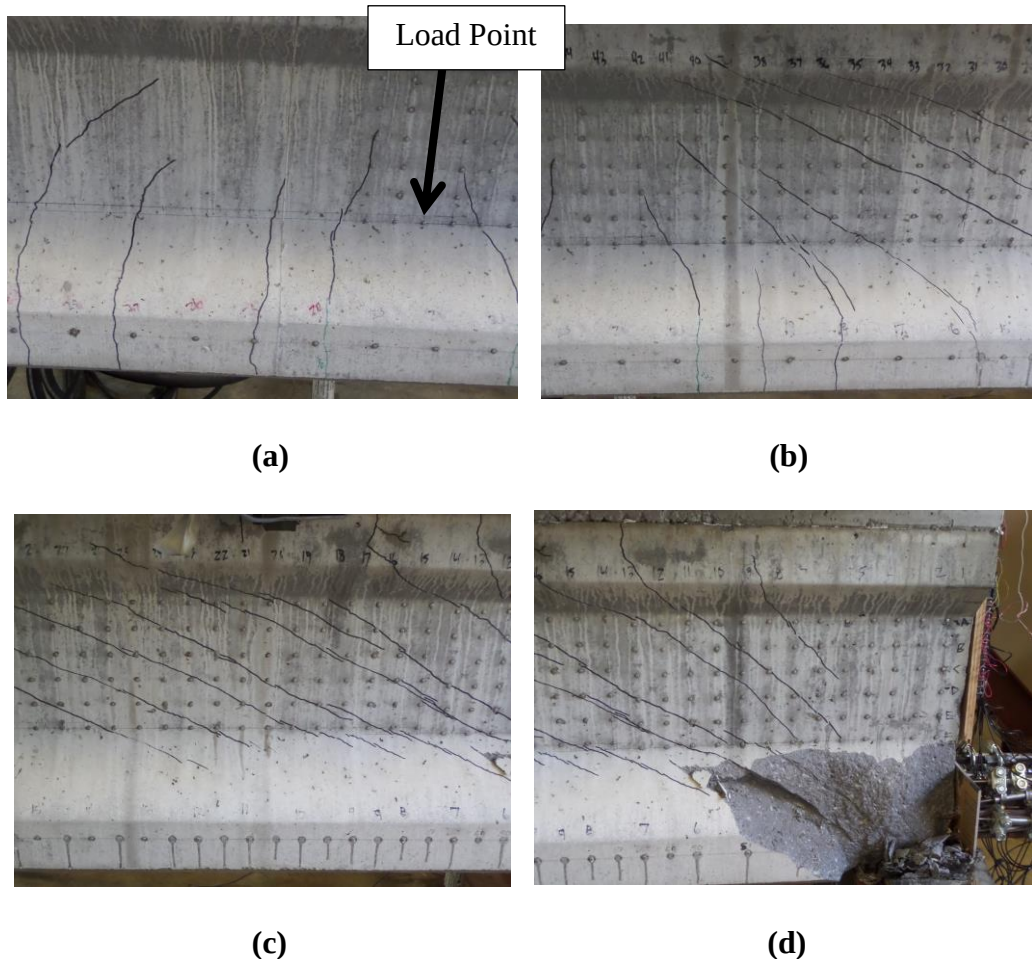


Figure 4.26 – Colleague of End Zone Cracking

The next load step was 235 kips. Continued web and flange cracking occurred though the web did not crush. The cracks were more prominent and the crack widths increased.

Additional load was then applied to the specimen in an attempt to reach a loading of 300 kips. At a load of 251 kips, several loud popping sounds occurred. Based upon the strand LVDT readings, the bond between the strand and the concrete broke loose with the formation of several web shear cracks. The final load that was placed upon the specimen was about 285 kips. This was the residual force that the strands allowed before slipping. **Figure 4.26** shows photos of the cracks that formed on the unpainted side of the beam. Cracks for both sides of the beam are shown in **Figure 4.27**.

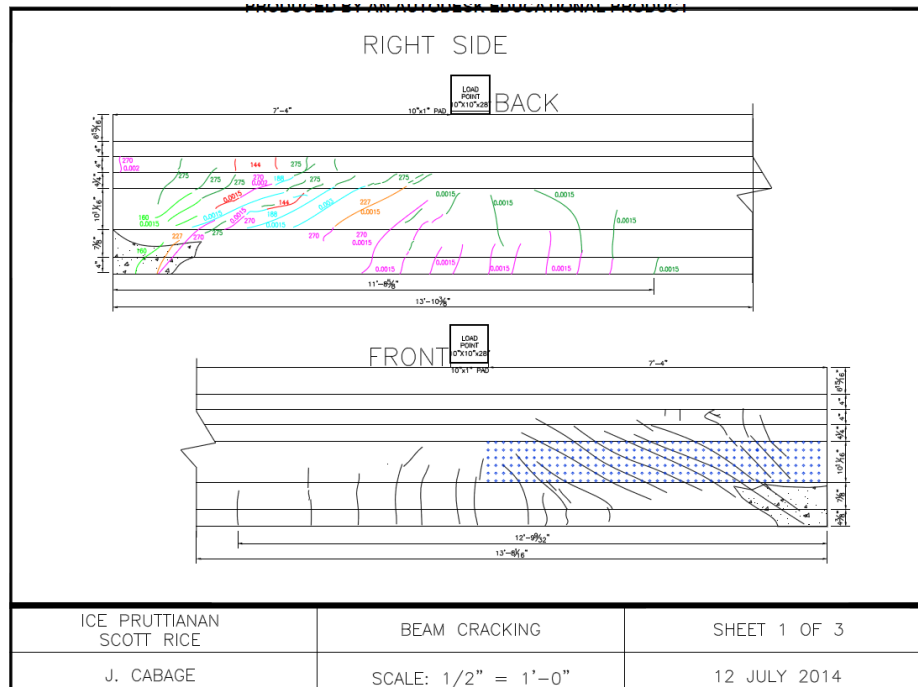


Figure 4.27 – AutoCAD Drawing of Test 0.70A Crack Pattern

The photos in **Figure 4.26** are in a series with the interior of the beam being shown in **Figure 4.26 (a)** and progressing towards the end with **Figure 4.26 (d)**. The load point is at the right side of **Figure 4.26 (a)**. The cracks at the load point are relatively vertical and begin to radiate in the web. In **Figure 4.26 (b)**, the cracks are about 45° through the web and become vertical in the flange. As the section is thickened in the flange, the shear stress is less prominent relative to the moment. As a result, the cracks become more vertical. In **Figure 4.26 (c)** the cracks in the web are more horizontal, with an angle between 22 and 28 degrees. Precompression due to release of the strands, a reduced moment influence, and the influence of a high shear load contributed to this crack pattern. In **Figure 4.26 (d)**, the cracks are more vertical in the web due to the reaction force at the support. The cracks were about four inches apart in the web of the beam.

One interesting point was the formation of cracks at the support. Typically, the compression from the reaction forces at the support is sufficient to prevent cracking in this area. In beam design both AASHTO and ACI allow that shear design be taken as the shear at a distance away from the support.

This specimen failed prematurely due to strand slip. The end anchorage region was severely disturbed and as a result the strand bond mechanism was changed due to the cracking. The cracks particularly in the end region would allow for a slip plane that would relieve the stress being transferred to the concrete, thereby changing the precompression regime. Anchorage would then be dependent upon bond which was shown in tests by Jiang, Ma, and Cabage (2013) to be substantially reduced at release.

Though the strands slipped and the specimen had severe web cracking, and the bottom flanges were shattered, the specimen held a residual load of about 250 kips. The average strain in the strand at 235 kips at the load point was measured to be 10,894 $\mu\epsilon$. The strand had reached its yield strength before the strand slipped, but full moment capacity was not reached in this test.

Following the test, the load frame was relocated and the support at the tested end was moved in so that the distance from the other end of the beam to the center of the support was 45.33 feet. This gave a 45 feet center-to-center span when testing this end of the beam. The damaged portion of the beam was cantilevered over the support.

For the second test, the strands which extended beyond the beam were hot bent and turned vertically. An attempt was made to cold bend the strands, but the 0.70-inch strand was too stiff to bend in this manner. Formwork was constructed and the strands were cast in a concrete block that was placed at the end of the beam. **Figure 4.28** shows the bent strand within the formwork and the casting of the concrete. A 4,000 psi concrete mix was batched and placed. The mix design is shown following **Figure 4.28**. The concrete was vibrated in place though care was taken not to vibrate near the strain gauges that were affixed to the strand. The concrete block was allowed to cure for 4 days prior to testing.

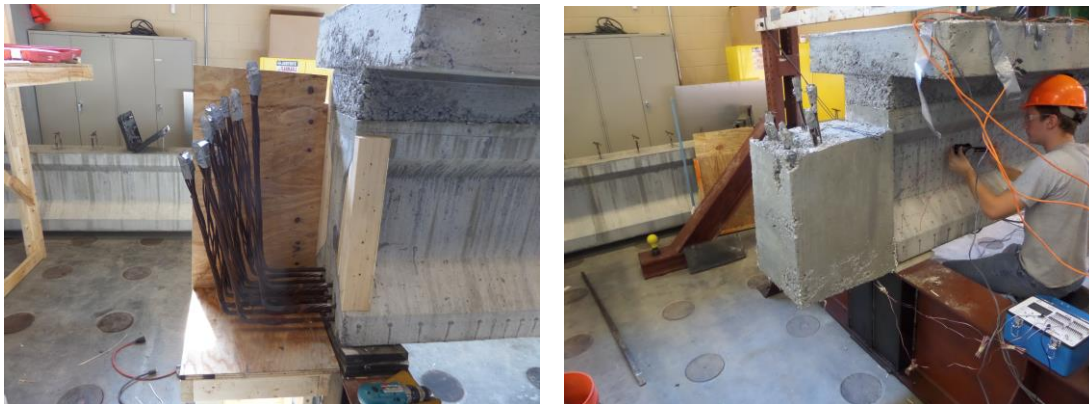


Figure 4.28 – Bent Strand within Formwork

End Block Mix Design – 5 cubic feet

Component	Weight Used (pounds)
Cement	110
Course Aggregate	320
Fine Aggregate	244
Water	48

Since the bearing pad failed in the first test, the neoprene was replaced with a cotton duck bearing pad. Testing of Specimen 0.70B began on June 23, 2014 and was completed on the next day. A load of 140 kips was placed on the specimen for the first load step. The specimen did not crack at this load step. Physical measurements were taken and recorded. The load was then

increased to 232 kips. The first crack in the web of the specimen occurred at a load of 170 kips. The test was paused while these cracks were marked. **Figure 4.29** is a photo of the cracking which occurred at an applied load of 285 kips.

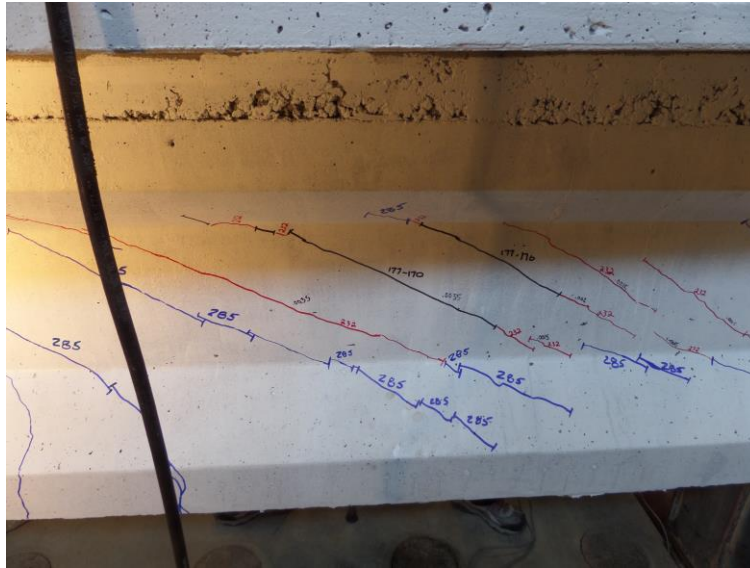


Figure 4.29 – Web Cracks on Specimen 0.70B at a Loading of 285 kips

The crack pattern for this specimen and specimen 0.70A was the same. The load at cracking was 22 percent higher for the second test. The crack patterns for both beams were similar and the cracks did not change direction with increased loading. This is not a new observation, but somewhat validates the use of the ratio of moment and shear in Response 2000. .

After the measurements were taken at this load step, the test was paused and continued on the next day. At that time, a load of about 350 kips was attempted. This was accomplished and a load of 356 kips was applied to the specimen. This load was near failure. Response 2000 predicted that the beam would hold a load of 374 kips as a maximum and AASHTO/ACI methods predicted a load of 400 kips. A time of about 10 minutes was given to allow the load to stabilize before measurements were taken. During this time, a loud concrete crushing sound came from the lower part of the beam at about four feet from the end of the beam. The concrete had crushed at the lower part of the beam. Once this occurred a loud bang occurred when one third of the composite deck on the painted side sheared off and fell to the ground. The concrete beam twisted under the eccentric load caused by the shearing of the deck. The side supports held the beam in place. **Figure 4.30** is a photograph of the failed beam.



Figure 4.30 – Photograph of Damage Beam

Figure 4.31 shows a close up of the damaged area at the top of the beam. The top flange and composite deck was severely crushed at the final load. It was difficult to tell whether the concrete flange crushed first or the concrete in the bottom portion of the beam released first.



Figure 4.31 – Close up of Damaged Top Flange at Load Point

The load was removed from the beam. The beam was uprighted using the overhead cranes. After it was clean and safe, the cracks were traced and recorded for this load stage.

Several interesting crack developed during this test. A photograph of this crack pattern is shown in **Figure 4.32**. The traced cracks in green show the cracking that occurred at the final load. In addition to web shear cracking, additional cracks were formed between the web shear cracks. These cracks altered the physical properties thus weakening the beam. This was the first time in both the small- and full-scale tests that this type of cracking was noticed.

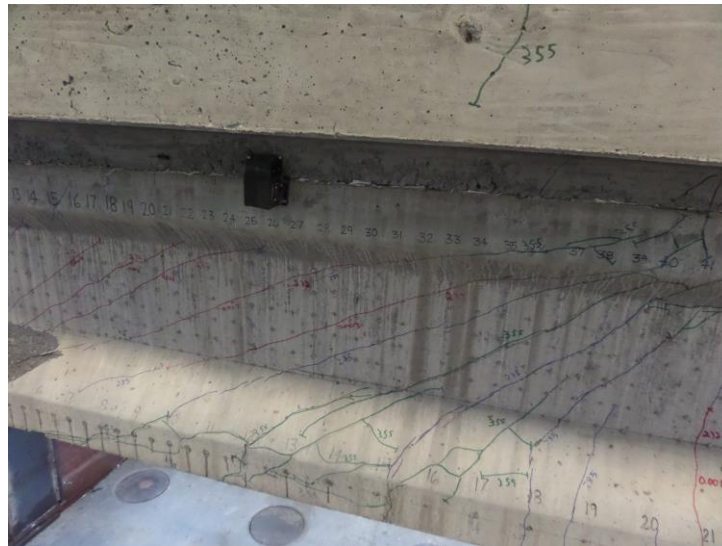


Figure 4.32 – Cracks which Formed at Final Loading

A second crack of interest was the cracks that formed at the strand block that was poured to encase the strand extensions, **Figure 4.33**. The concrete block was poured with direct contact at the end of the beam. At the 285 kip loading stage, a separation at the top of the block and the beam began to occur. As the load was increased, the crack began to widen. The bottom of the block and the beam remained uncracked during this test. This indicates that the block took some loading from the strand during this test, but the strands remained in place.



Figure 4.33 – Crack Formed by Rotation of End Block

Again, the bottom flange on one side of the beam cracked and sheared off. This occurred at final loading and may have been caused by the twisting of the beam. **Figure 4.34** shows the sheared bottom flange. Note the formation of shear cracks throughout the end zone of the beam. The cotton duck bearing pad performed well in this test and added to the overall performance of the beam. The pad did not collapse and seemed to better distribute the loads to the support. The painted side flange did not shear off nor did the sides shear off until the beam began twisting. This twisting motion would concentrate the bearing to the unpainted side causing a higher shear stress at the heaviest applied load.



Figure 4.34 – Shearing of Bottom Flange at Final Loading, Specimen 0.70B

As before, vertical cracking also occurred in the top flange of the beam at the end of the beam. This is shown in **Figure 4.35**.



Figure 4.35 – Vertical Top Flange Cracks, Specimen 0.70B

These cracks became apparent as a hairline crack at a loading of 285 kips. The cracks continued to expand and had a separation of about $1/16^{\text{th}}$ of an inch at final loading.

The final cracks of interest occurred within the deck itself. Since the concrete shear off, many of the cracks that formed in the deck were exposed. The concrete strength in the deck at the time of test was approximately 10,600 psi. Even with this high-strength concrete, cracks formed within the deck under loading. The first crack of interest is the crack that formed directly underneath the load point, **Figure 4.36**. The crack was initially noticed at a load of 285 kips. At failure, the crack had expanded to a width of approximately 0.20 inches. This shows that, at final load, the composite deck was separating from the AASHTO beam. In other words, the horizontal shear connectors were not sufficient to resist the forces applied to them.

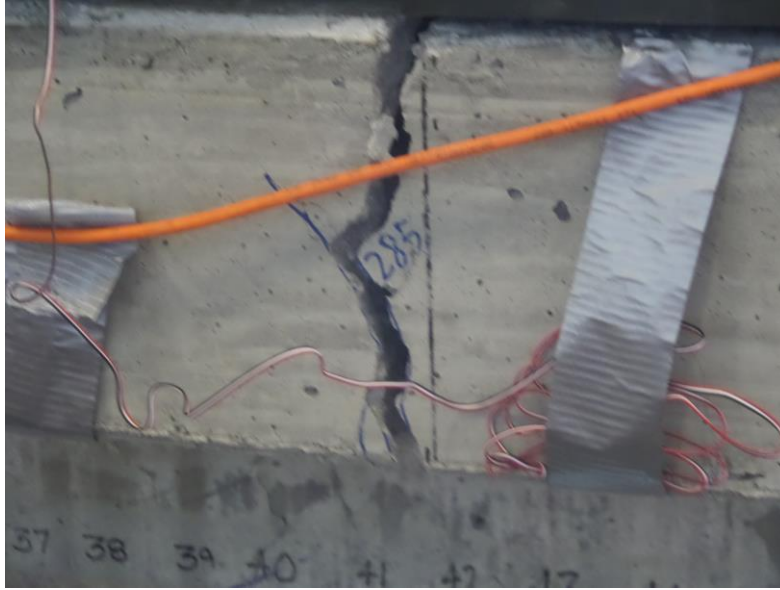


Figure 4.36 – Crack Formation in Deck Directly under Load Point

The forces on the shear connectors were a combination of tensile force, from acting as stirrups, and a shearing force from the horizontal force applied by composite action. Several of the vertical shear connectors were inspected after the test by removing the concrete surrounding them. They did not appear to show signs of lateral deflection. **Figure 4.37** shows a vertical shear stud and the cracking in the deck nearby. The deck was severely cracked in many places from the load point in both directions, but particularly towards the end of the beam. It could be that as the crack at the load point was forming that the slippage noticed there was compensated by the formation of these cracks.



(a)



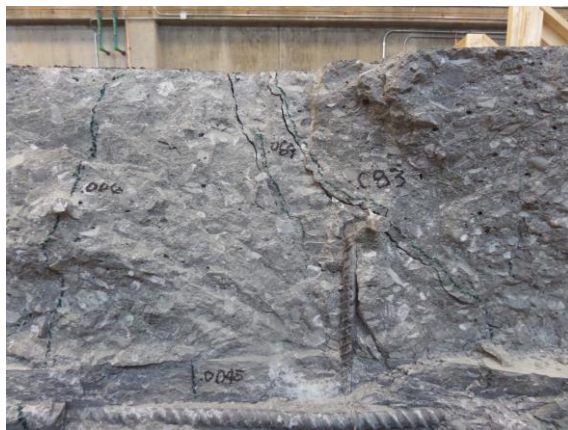
(b)



(c)



(d)



(e)



(f)

Figure 4.37 – Cracking in the Top Deck, Specimen 0.70B

After the test was completed and the results were recorded, the load frame was removed from the over the beam and placed in another part of the lab. The beam was lifted with both 10 ton overhead cranes and placed upon an extended trailer then shipped to Britton Bridge, LLC's yard to be used for additional testing and a test specimen for impact testing. **Figure 4.38** shows the beam loaded upon the truck and leaving the high bay structural lab.



Figure 4.38 – Loading of First Bridge Beam on Truck

4.7 Results from Instrumentation Data

The load-deflection behavior is shown in **Figure 4.39**. Four plots are presented, one for each day of the test.

For Test 0.70A, the behavior was basically linear to a load of about 235 kips. Load was increased after measurements were taken at the 235 kip load step. The load was increased until the bond between the strand and the concrete broke. The shape of this plot above the 235 kip load step had a similar shape to the plots from the testing of Specimen 0.70B.

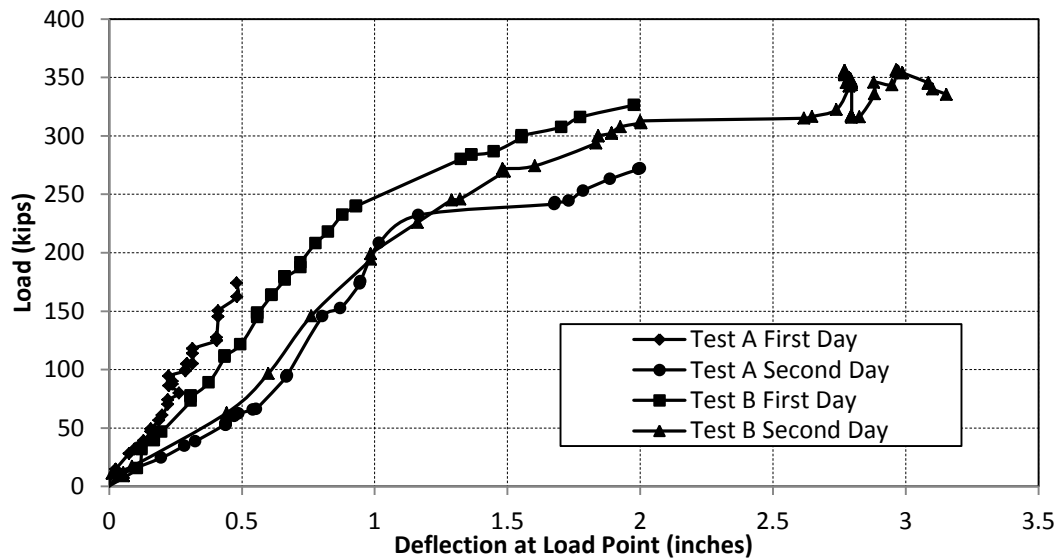
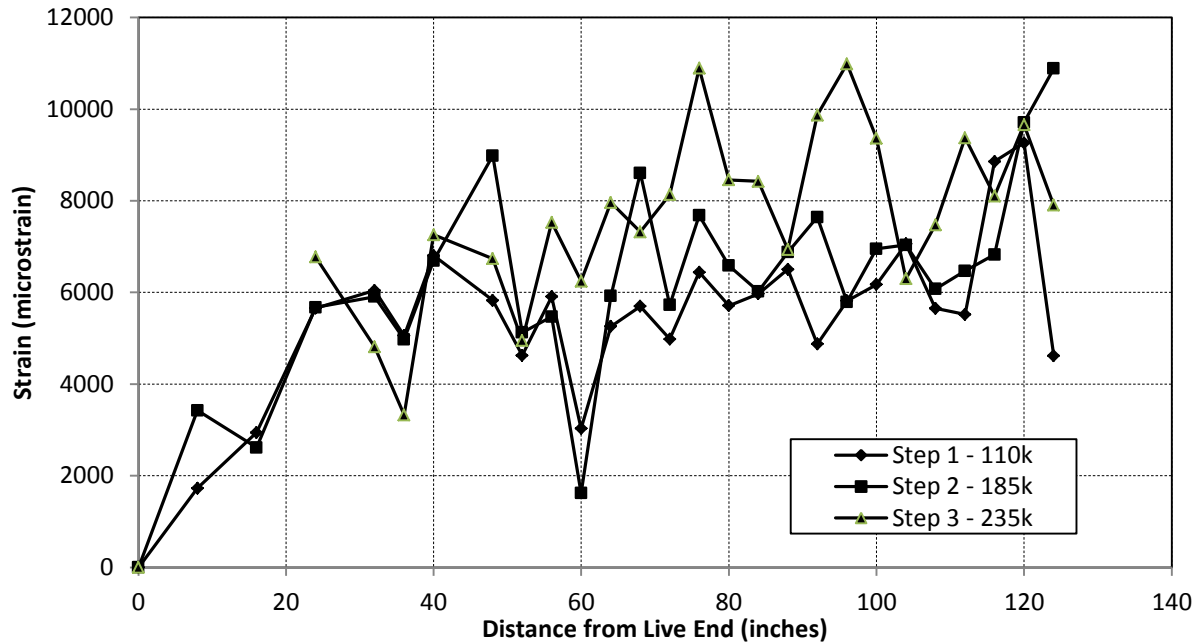


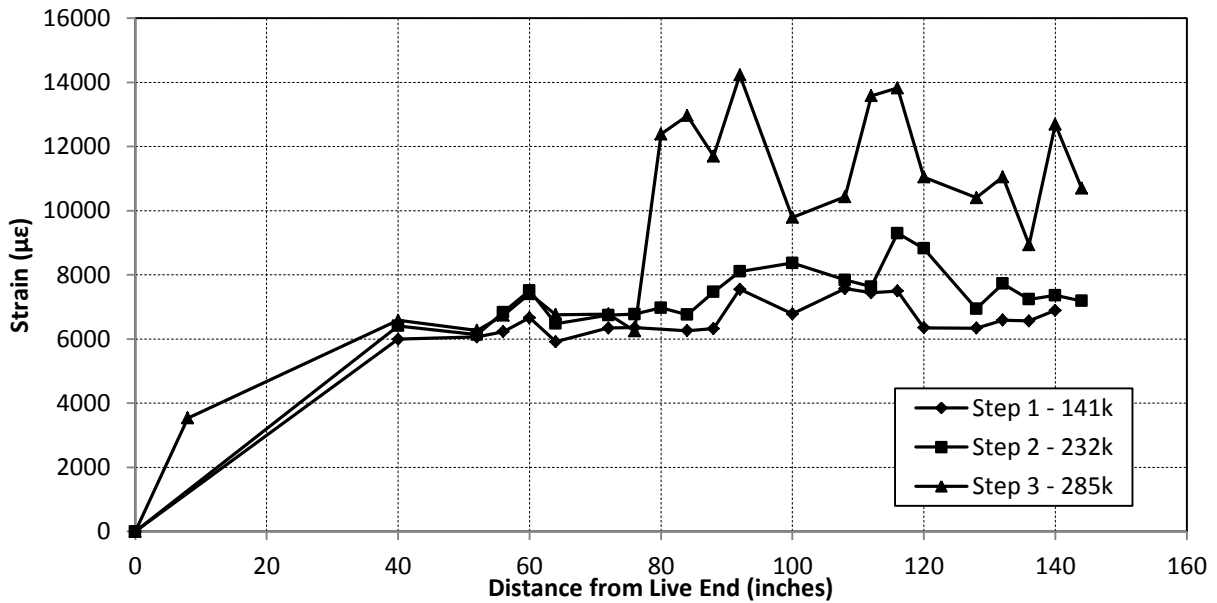
Figure 4.39 – Load versus Deflection 0.70-inch Strand Beam Tests

The behavior for Specimen 0.70B was linear to a load of about 230 kips and then began to break over at that point. It appears that the strand began to yield at this point. The specimen was able to take more load and at the end of the first day of testing did not appear to be in the plastic range. On the second day, a load of 350 kips was attempted. In retrospect, a load of 310 kips would have been more desirable as the specimen began to behave plastically at that point. It would have been nice to have been able to record the strain measurements from that point. A 350 kips loading level was attained. The MTS actuators were used to hold this loading level, but as this loading was stabilizing, the bottom flange of the beam crushed and the beam failed.

Figure 4.40 plots the concrete surface strain at the strand centroid from the end of the beam for Specimen 0.70A and 0.70B are presented. An estimate of the initial prestressing strain was added to the measured results. The initial strain was estimated to be the strain at release with a 20 percent reduction for time-related losses.



(a) Test 0.70A



(b) Test 0.70B

Figure 4.40 – Concrete Surface Strain Plots at Strand Centroid

The maximum strain was 10,894 microstrain at the load point for Test 0.70A. This was 87 percent of the yield strain of 12,500 microstrain. This indicates that the specimen was close, but

not at the yield strength of the strand before slipping. The maximum strain was 14,228 microstrain at the load point for Test 0.70B, providing further evidence that the strand had yielded.

Figure 4.41 shows the relative movement across the beam/haunch interface. One end of the specimen was designed for shear using AASHTO procedures, the strong side, and the other using strut-and-tie modeling procedures, the weak side. The strong side was used for Specimen 0.70A and the weak side for Specimen 0.70B. The vertical shear spacing was six inches on center within the end zone region for the strong side and eight inches on center for the weak side.

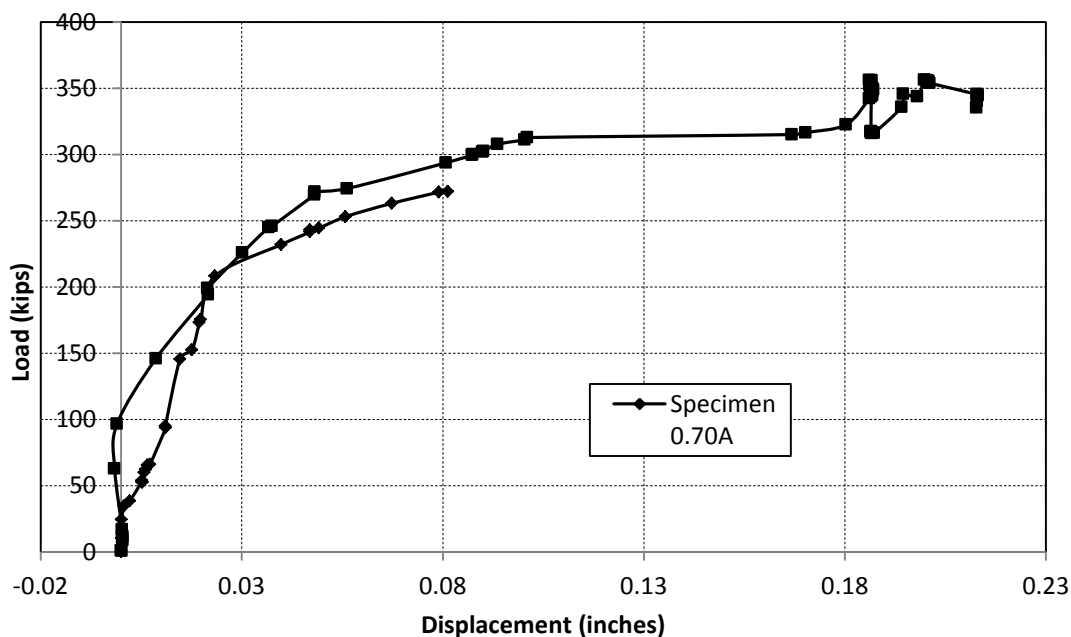


Figure 4.41 – Relative Lateral Movement across Beam/Haunch Interface at 46 inches from End of Beam

As the load approached 300 kips, the interface continued to move without the addition of load.

From **Figures 4.39 (e) and (f)**, it can be seen that the vertical reinforcement did not move horizontally, but that the concrete that surrounded the vertical reinforcement cracked. It was believed that the concrete cracked and moved without causing appreciable lateral movement of the vertical shear reinforcement.

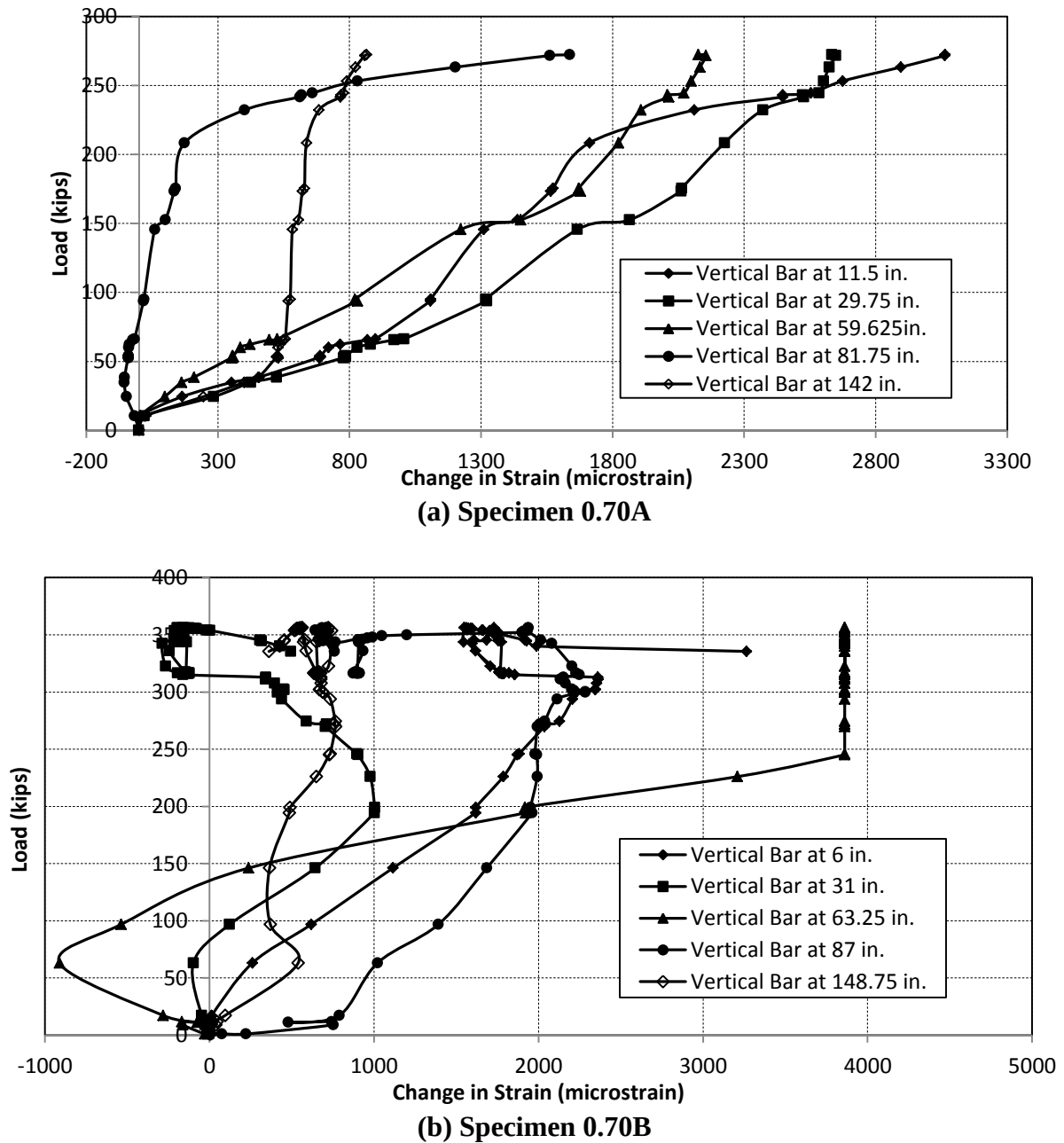


Figure 4.42 – Strain in Horizontal Shear Reinforcement

The load versus strain in the vertical reinforcement plot is shown in **Figure 4.41 (a)** and **(b)** for both tests. The yield strain of the rebar can be taken as 2070 microstrain. The strain gauges indicate that the first three vertical bars had yielded under a 200 kip load. The gauges for Specimen 0.70B show an erratic behavior above 300 kips. This could be a result of the strain gauges shearing from the rebar or being extended beyond their capacity.

During this testing, it was found that the formation of cracking in the end zones allowed premature strand slip failure. This was demonstrated in the small-scale tests. When the formation of the thrust crack developed, the properties of the beam changed and the concrete frictional bond was broken. This was generally followed by an immediate strand bond slip failure. Bending strand and encasing them in concrete at the end of the beam has been shown to anchor the strands even when cracking in the end zone exists.

This phenomenon was also seen in the full-scale testing. Test 0.70A failed prematurely as cracking formed in the end region and the strand slipped. A concrete block was poured around the strand ends for Test 0.70B. Strain gauges were attached to these strands and a specific response was noted as loading was increased. This indicates that the strand slip response was repeatable, but the encasement of the strands prevented the strands from slipping on the second test. This allowed a much higher loading capacity and f_{ps} strain was achieved during the second test with a concentrated load applied at 15 percent of the span length.

4.8 Summary of Full-Scale Tests

4.8.1 Evaluation of Shear Resistance

Several different methods were used to determine the shear resistance of the AASTHO Type I beam. One end of the beam was designed using strut-and-tie modeling—the weak side—and the other using AASHTO equations—the strong side. In addition, Compression Theory, Modified Compression Theory, and the Response 2000 Program were used to evaluate shear resistance. The beams were designed with two #4 vertical reinforcing placed at 8 inches on center on the weak side and 6 inches on center on the strong side.

The second test was performed on the beam end designed using the strut-and-tie model. This end was subjected to the highest shear load during testing. The weak side was tested to a load of 356 kips—about 315 kips shear—without causing a shear failure. **Table 4.4** provides the shear capacity for both steel and concrete using the models described in the dissertation. All models underestimated the actual shear capacity of the beam. The beam held a shear of 315 kips which is about four percent above the capacities predicted using Response 2000 and the AASHTO Simplified procedures. Based upon crack observations, it was believed that shear failure was impending.

Table 4.4 – Shear Capacity of Composite Beam

Full-Scale Test	0.70A			0.70B		
	V_c	V_s	V_{total}	V_c	V_s	V_{total}
Stirrup Spacing	8 in.			6 in.		
Prediction Model						
Modified Compression Theory	49k	194k	234k	49k	146k	195k
AASHTO General Procedure	100k	233k	333k	100k	175k	275k
AASHTO Simplified Procedure	128k	239k	367k	128k	179k	307k
Response 2000			307k			301k
Actual Test Results			254k			315k

4.8.2 Moment Capacity and Flexural Cracking Predictions

The moment capacity for this beam was estimated to occur with a loading of 400 kips (ACI/AASHTO prediction). The maximum loading at failure was 356 kips. Based upon our observations and analysis, the full moment capacity was not realized due to the failure of the beam due to excessive displacement at the top of the beam and the haunch. A horizontal shear failure did occur as noted in the deck separation shown in **Figure 4.34**, but the concrete was beginning to crush at the load point and at the lower flange of the beam as presented in **Figure 4.30**.

One observation was that the Response 2000 did an excellent job of predicting the crack shape, type, load, and angulation of this test. The first cracks occurred in each test at a load of 144 kips for Specimen 0.70A and about 160 kips for Specimen 0.70B. Response 2000 predicted cracking to begin at 138 kips. This cracking may have occurred at that loading level, but was not observed until a little later. The Response Program anticipated cracking at yield, when yielding was observed within the beam, and also predicted a maximum beam capacity loading of 386 kips.

It is believed that this capacity would have been attained provided there was additional horizontal shear reinforcement placed in the beam. The behavior of the beam and the behavior predicted by this program were uncanny. This result demonstrates two things. The first is that the design equations currently used to design beams using AASHTO can be used to design

beams using high-strength concrete and larger diameter strand. And second, modified compression field theory is valid for beams using 0.70-inch strand and high-strength concrete.

4.8.3 Strand Slip Resistance and Tension-Tie

The full-scale test did not prove to be adequate to evaluate tension tie resistance. The strands did slip at a load of approximately 274 kips and, based upon strain gauge measurements within the strand concrete encasement block poured at the end of the beam would have failed a second time at this loading. These strain measurements may indicate that the development or tie length of the beam should be greater than seven feet where the small-scale tests and the modeling work suggest much smaller lengths.

The major difference in the two series of tests is the development of shear cracking at the end zone. Under this high shear load, the cracking extended throughout the end zone region. This type of cracking has been shown in this series of tests to hinder the ability of the concrete to bond to the reinforcing strand. The bonding mechanism in strand uses frictional forces developed at strand release to anchor the strand to the concrete.

Under normal/uniform loading conditions, the shear demand is considerably less than the demand required by loading in these tests. For instance, if a uniformly distributed load is used to develop a 2,290 k-ft moment capacity at the center of a 45-foot span, AASHTO Type I beam, then the unfactored reactionary shear force in this system would be 35.8 kips. This is calculated using a dead load of 0.59 kip/foot and a live load of 1.0 kip/foot. A shear force of 35.8 kips is well below the cracking force of 138 kips predicted by Response 2000 and the 144 kip cracking force actually observed during testing.

Though this level of shear loading is not normally used in beam design, the transfer and development lengths of the larger strand are much shorter than intuition would expect. As a result, the forces developed at release will be distributed to the beam much quicker than may be expected. Because of this, additional care should be given in examining cracking in the beam at release to ensure that no cracks are formed in the end zone of the beam at any stage whereby the bonding mechanism of the strand may be adversely impacted.

4.8.4 Bearing Pad Design Specification

The first beam failure mechanism that was observed in these tests was that as web shear cracks were forming at the end of the beam, a shearing occurred at the bearing location of the bottom flange to the web. After the first test, it was assumed that this occurred because the neoprene pads used in the first test failed, causing the entire reactionary load to be concentrated at the edge of the support. The difference in the stiffness of the flange and the web would allow a differential deflection to occur which would then generate a shear force which was resisted by the shear plane between the web and the flange.

In those test specimens, embedded steel reinforcement does not cross this plane. As a result, only the strength of the concrete was available to resist this shearing force. In both full-scale tests, shearing occurred. In the second, test a cotton duck bearing pad was used. During the second test, a bearing pressure of approximately 2400 psi was applied at the support. The cotton duck bearing pad performed well under this load even though at the end of the test the bottom flange of the second test also failed.

As a design consideration, it is recommended that the vertical steel reinforcement be detailed so the reinforcing steel crosses this shear plane. This is particularly important for beams with high shear loads being applied to them. Also, the use of cotton duck pads proved to be effective in this testing whereas neoprene pads did not resist the loads applied during this testing.

4.8.5 Beam Response using Larger Diameter Strand and High-Strength Concrete

In completing this work, it has been found that the current models predict the capacity of a beam using high-strength concrete and larger diameter strand. The beam did perform in a predictable manner even with the placement of a high concentrated shear load at the end of the beam. This type loading was used because it represented an extreme case scenario whereby the beam could be vetted for future use as a bridge girder. The predictions of beam behavior predicted with the Response 2000 program proved to be accurate. The predictions of moment capacity were a little less than expected. This was likely due to inadequate horizontal shear resistance in the beam.

Because the beams behavior was accurately predicted using existing beam modeling procedures, this type of system appears to be viable for design. As a result, it is the recommendation of this study that beams using larger diameter strand and high-strength concrete can be used in bridge girders.

When designing one of these beams, it is important that steel reinforcement extends across the vertical shear plane between the web and flange of the beam. This testing has shown that a fracture plane can be developed across this plane when the bearing pressure exceeds 1,000 psi. High shear loads are generally not typical of bridge girders. This being the case, it is still prudent to ensure that the combined shear and moment loading of a beam does not cause end zone cracking. This series of tests has shown that cracking in the end zone is detrimental to strand bond. This concept is also important when debonding strands within a beam. It is important to realize that as the debonded strands are developing their longitudinal strength, flexure and shear cracks can disturb the bond development. As a precaution, cracks in an area of strand bond development should be prevented either by design change or section alteration to avoid this situation.

In the test 0.70B, the strands were turned upward to hinder a bond slip failure. This proved to be very effective in preventing strand slip. The fact that the strands were bent upward kept the strand from twisting thereby increasing the mechanical interlock capacity of the strand to concrete bond. Full moment capacity would have been achieved in this test without strand slip if the specimen had not failed in horizontal shear.

Numeric models that were used in this investigation—particularly the Response 2000 Program—performed well when using high-strength concrete and larger diameter strand. The reinforcement file in the program was modified to include the larger diameter strand, and once they were in place, the model can be used effectively for parametric studies involving this material.

In this work an iterative numeric model was developed for strand transfer and development length prediction. The procedure worked and demonstrates that a frictional analysis predicts that larger diameter strand will have a shorter development length. The model did a much better job of predicting development and transfer lengths than the current prediction models.

If current AASHTO transfer and development length formulas are used, end zone stresses will be underdesigned. The AASHTO equations predict a much longer transfer and development length. From a beam moment capacity standpoint, this allows a longer distance for the strand to develop which is conservative, but severely underestimates the transfer and development stress in the end zone region. Ideally, the shorter distance should be used for strain transfer and release stress calculation and a more conservative distance for moment capacity.

4.8.6 Constructability of Beams with Larger Diameter Strand

During the course of this investigation, two full-scale specimens and six small-scale specimens were prepared at Ross Prestress Concrete, Inc. Interviews were conducted with the workers and management of the company to get their opinion of the ease of construction and the different modifications that were necessary to use this strand. One significant advantage of this strand is that a beam of the maximum delivery length in the State of Tennessee can be manufactured and delivered without changing the forms at the plant.

The larger diameter strand is much stiffer and heavier than the 0.50-inch strand. The strand was heavier to drag across the beds and a little more difficult to insert into the formwork at the end of the beds. Larger holes in the steel plate formwork had to be drilled to accommodate larger strand. If the 0.70-inch strand becomes of prominent use, then the plant management felt that the counterweight and the jacking blocks needed to be upgraded.

Bending the strands upward proved to be very effective in preventing premature bond slip of the strands. To bend the 0.70-inch strand, the strand had to be heated. Smaller strand can be cold bent, but the larger strand had to be heated. When heating this strand, it was found that it was easy to apply too much heat thus melting the strand. Initially the strands were bent at the 0.70A side of the beam and several of these strands were broken in the process. The bent portion was cut off, and later the strands were bent on the 0.70B side and encased in concrete. A lesser amount of heat was used on each strand on the 0.70B side to bend the strand without causing breakage. Bending the strand can be conducted very easily after some practice.

Based upon interviews with manufacturing plant management and employees, larger diameter strand can be used readily without major plant modifications and upgrades to their equipment.

5.0 CONTRIBUTIONS OF THIS WORK

This dissertation examined development length in larger diameter strand. Prestressed larger diameter strand has a shorter development and transfer length than current models predict. A modeling procedure was developed for prestressing strand that predicted that the transfer and development length would be shorter with larger diameter strand when a frictional analysis was performed.

Two full-scale beam tests were conducted. Tests results were compared to exiting behavioral models for moment, shear, and cracking. The behavioral models, particularly Response 2000, predicted the behavior of the beam very well.

Beam constructability with larger diameter strands was examined. It was found that beams with larger diameter strands could be constructed with minimal plant upgrades.

Based upon the findings of this research, beams with larger diameter strands can be successfully manufactured and used for bridge girders. This will allow for the efficient use of high strength concrete without having to replace current formwork.

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APPENDICES

Appendix A – Strand Material Certifications and Other Information



INSTEEL WIRE PRODUCTS®

MATERIAL CERTIFICATION OF COMPLIANCE

Sanderson, FL

ROSS PRESTRESSED CONCRETE INC
2701 INDEPENDENCE LANE
KNOXVILLE TN 37914

Bill of Lading: 00112919
Order Number: 386673
PO Number: verbal

Insteel Wire Products Company hereby certifies that the specimens taken from production lot(s) consisting of one or more of the following Lot/serial numbers were tested in accordance with and met the specification requirements of A 416 – 10. The attached test report(s) represent the result of such test(s).

.700 (17.8mm) 7W 270 LR ASTM A416

Test Report Number: 10073690

Lot/Serial Numbers

1303462109036
1303462109037

Heat Number/Lot Number

13239

The products listed in this certification were manufactured and fabricated in the United States of America.

Insteel Wire Products Company hereby certifies that the prestressing strand described above meets or exceeds the minimum bonding requirements as currently accepted in the NASP (North American Strand Producers) pull-out test and the Moustaffa block pull-out test.

All Domestic Prestressing Strand was made from steel rod that was manufactured and processed completely in America. The rod was then manufactured into PC Strand in the United States at Insteel Wire Products Company plant in either Jacksonville, FL, or Gallatin, TN. The material meets the "Buy America" requirements of 23 CFR 635.410.

The products listed in this certification were manufactured in the U.S.A. from wire rod which was manufactured in the U.S.A.

Quality Assurance Manager: _____

Channing R. Reynolds Jr.

Date: 27-AUG-2012

INSTEEL WIRE PRODUCTS TERMS AND CONDITIONS

1. **Carrier Compensation:** Shipper agrees to pay Carrier's compensation for transportation for services rendered at the rates provided for in Exhibit "A" of the Transportation Agreement and or Exhibit "B" of the Contract for Brokerage Transportation. Carrier shall promptly issue a freight bill to the shipper in care of

Insteel Wire Products Company
1373 Boggs Drive
Mount Airy, NC 27030
C/O AP Traffic

Payment shall be due within thirty (30) days after the date of the invoice providing Carrier submits required paperwork with invoice.

2. **Delivery:** Truck shipments are based on applicable state limit truckload lots. Deliveries shall be made in accord with a delivery schedule, which may be revised to adjust manufacturing requirements. Acceptance of delivery constitutes your acceptance of the delivery schedule against which the delivery was made.
3. **Warranty and Limitations:** Except as specifically agreed in writing by Insteel Wire Products, the products furnished hereunder shall be produced in accordance with Insteel Wire Products standard practices. All products, however, including those products to meet exact specifications, shall be subject to mill tolerances and variations consistent with good mill practices with respect to dimension, weight, section, composition and mechanical properties, and to normal variations in surface and internal conditions and in quality. Products shall also be subject to variations from tolerance, and variations consistent with practical testing and inspection methods, and to regular mill practice on over and under shipments. INSTEEL WIRE PRODUCTS MAKES NO WARRANTY, EXPRESS OR IMPLIED EXCEPT THAT THE PRODUCTS SHALL BE PRODUCED IN ACCORD WITH THE SPECIFICATIONS ON THE FACE OF THIS ORDER. ANY IMPLIED WARRANTY OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE IS HEREBY DISCLAIMED BY INSTEEL WIRE PRODUCTS AND EXCLUDED FROM THE AGREEMENT. ALL MATERIALS ARE SOLD "AS IS" & THERE ARE NO WARRANTIES WHICH EXTEND BEYOND THE DESCRIPTION OF THE FACE HEREOF. Insteel Wire Products' responsibility under this Warranty is LIMITED as follows:

- A. To the repair or replacement of defective or non-conforming products or to the allowance of a credit for such products, at our option, strictly in accord with the procedure stated in Article 4-Claims.
- B. Under no circumstances shall our maximum liability for damages exceed the contract price for the products claimed to be defective or unsuitable.
- C. We shall not be liable for special, consequential, direct, indirect or incidental damages including without limitation any liquidated damages or penalties of any kind you may incur. We assume no obligation for expenses of any kind, whether arising from delays during replacement of materials for cause or otherwise.
- D. Any action for breach of contract, breach of warranty, negligent manufacture or otherwise, arising from this Agreement or from condition of materials described herein must be commenced within one year after the cause of action has accrued.
- E. **Secondary Material:** Notwithstanding any representations or warranties made above, if this Order Acknowledgement describes any materials to be furnished to you as "secondary", "sec", "as is", or otherwise describes our products as seconds or less than prime, then the following disclaimer shall apply: TO PRODUCTS DESCRIBED HEREIN ARE SOLD "AS IS" AND "WITH ALL FAULTS". INSTEEL WIRE PRODUCTS MAKES ABSOLUTELY NO WARRANTIES, INCLUDING NO WARRANTY OF MERCHANTABILITY NOR FITNESS FOR A PARTICULAR PURPOSE. ANY STATEMENTS MADE HEREIN THAT ARE INCONSISTENT WITH THE PROVISIONS OF THIS DISCLAIMER ARE INAPPLICABLE TO SALES OF SECONDARY MATERIAL. In no event shall Seller be liable in breach of contract, warranty, negligence, strict liability, or any other cause of action in contract or tort for personal injury or damages, or any consequential, direct, indirect or incidental damages. We make no representations about the product's condition, quality, character or about its ability to be used by buyer.
- F. THERE ARE NO WARRANTIES WHICH EXTEND BEYOND THE ABOVE AND OUR SOLE RESPONSIBILITY AS STATED. Any and all representations, promises, warranties or statements by Insteel Wire Product's agents that differ in any way from the terms of this agreement shall be given no force or effect.

4. **Claims:**

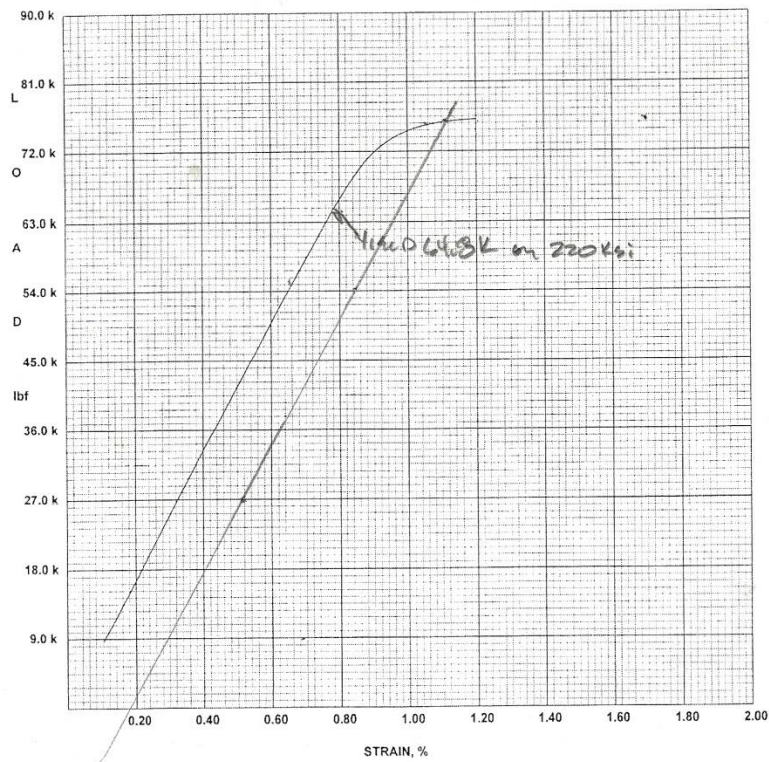
- A. Damages in transit: Shipments must be inspected by you before unloading to ascertain any damages enroute. Claims for shortages or damages must be made by notation on the face of the freight bill at the time of unloading. The carrier, when accepting materials at our plant is responsible for damages in transit and all claims for damages in transit shall be made promptly to the carrier who accepts our products for transportation.
- B. Claims for shortages or defective materials or non-conformity to specifications, which would be revealed by prompt inspection, must be made in writing to Insteel Wire Products immediately and in any event, within 5 days after you receive the material so that any such claims can be investigated promptly.
- C. Claims for defective materials or non-conformity to specifications, will be investigated promptly provided that you give us notice of the alleged defect or non-conformity in writing within 30 days after receipt of material. If the alleged defect or non-conformity to specifications is discernable from prompt inspection of the material you must advise us immediately upon receipt of the material.
- D. In any event, if you do not provide us with written notice of any defect or non-conformity within thirty (30) days after delivery, such a failure to notify us shall constitute an irrevocable acceptance of the materials and workmanship, a waiver, and admission that you have had full and complete opportunity to inspect the material, an admission that the materials and workmanship fully comply with all terms and specifications of this agreement, and you shall be stopped from claiming otherwise.
- E. No claim will be allowed except as provided above, and your remedies as provided herein and in paragraph 4 above are you exclusive and sole remedies.

5. **Force Majeure:** Insteel Wire Products may postpone this order as to undelivered shipments, and shall not be responsible or liable for any delays or non-performance in the event of fire, flood, explosion, the elements, other catastrophe, acts of God, war, riot, civil disturbance, strike, lockout, refusal of employees to work, labor disputes, shortage or inability to obtain raw materials, failure of carriers to deliver either machinery, equipment, or material, in the event of any legislative, executive or judicial act of any political or governmental authority substantially effects Insteel Wire Products.

6. **Miscellaneous:**

- A. Insteel Wire Products' failure to insist upon strict performance of any provision of this agreement shall not constitute a waiver of that or any provision of any of the sellers rights under this agreement, nor shall it constitute a waiver by Insteel Wire Products of any subsequent default by you in the performance of this Agreement.
- B. You agree to pay any and all costs, expenses, and attorney's fees which we may incur or become liable for by reason of our enforcing or attempting to enforce the terms of this Agreement.
- C. This agreement shall be construed & governed according to the law of the State of North Carolina, and any and all disputes arising hereunder shall be litigated in the State Courts of North Carolina.
- D. This agreement shall be binding upon and inure to the benefit of the respective heirs, executors, administrators, receivers, legal representatives, successors, and assigns of the parties.
- E. Should any part or provision of this agreement be declared invalid, unenforceable, illegal, or in conflict with any law, the validity and enforceability of the remaining portions or provisions or portions of provisions shall not be affected.

Yield = 75.6K
Yield Strain = 1.05%



Insteel Wire Products
Prestressed Concrete Strand

.700 270 7W LOW RELAXATION

Test Number:	10073690
Tested By:	CER
Ultimate Breaking Strength, lbf:	82413
Ultimate Breaking Strength, kN:	367
Load @ 1% Extension, lbf:	74603
Load @ 1% Extension, kN:	332
Ultimate Elongation, %:	5.73
Representative Area, in ² :	0.294
Representative Area, mm ² :	189.7
Actual Area, in ² :	0.2923
Actual Area, mm ² :	188.5608
Avg Modulus of Elasticity, Mpsi:	29.0
Avg Modulus of Elasticity, MPa:	199947.6
Reference:	

Aug 27, 2012 12:34:44 PM
SN: 206250-R2 V7.02.07



INSTEEL WIRE PRODUCTS

Sanderson Plant - Sanderson, FL 32087

Packing List

ROSS PRESTRESSED CONCRETE INC
2701 INDEPENDENCE LANE
KNOXVILLE TN 37914

Bill of Lading Number: 00112919

Page: 1 of 1

Date: 27-AUG-12

Sales Order Number: 386673

Customer PO Number: verbal

Lot Number	Description	Length	Weight
1303462109036	.700 (17.8mm) 7W 270 LR ASTM A416	3360	3365
1303462109037	.700 (17.8mm) 7W 270 LR ASTM A416	3360	3365
Total Number of Packs 2		Totals	6720 6730

This Shipping Order

must be legibly filled in, in Ink, in indelible Pencil, or
in Carbon, and retained by the Agent.

CUSTOMER COPY

Page 1 of 1

RECEIVED, subject to individually determined rates or contracts that have been agreed upon in writing between the carrier and shipper, if applicable, otherwise to the rates, classifications, and rules that have been established by the carrier and provided via certified mail to the shipper.

The property described below, in apparent good order, except as noted (contents and condition of contents packages unknown) marked, consigned, and destined as shown below, which said carrier agrees to carry to destination, if on its route, or otherwise to deliver to another carrier on the route to the destination. Every service to be performed hereunder shall be subject to all the conditions not prohibited by law, whether printed or written, herein contained, including the conditions hereof, which are hereby agreed to by the shipper and accepted for himself and his assigns.

FROM: **INSTEEL WIRE PRODUCTS**
1 Wiremill Road
AT: Sanderson, FL 32087
(Mail or street address of consignee - for purpose of notification only.)

CONSIGNEE AND DESTINATION: **ROSS PRESTRESSED CONCRETE INC**
2701 INDEPENDENCE LANE
P O BOX 6299
KNOXVILLE TN 37914

DATE: 27-AUG-12
SHIPPER'S NO.: 00112919-3781173
STOP OF:

BY CARRIER: PREFERRED PINE STRAW INC

BROKERED SHIPMENTS:
The Broker may bill the shipper directly on behalf of the Carrier and payment thereof shall relieve the Shipper of any liability to the Carrier for non-payment.

Hazardous Material	No. Pieces/Units	Description of Articles, Special Marks and Exceptions	Sales Order No.	Unit Wt/Length	Weight
	6720	59B-051653 .700 (17.8mm) 7W 270 LR ASTM A416 PO: verbal Please ship 2 coils at 3,360 feet each to Ross Prestress for The University of TN.	386673	1	6731
		Total Pieces: 6720			
		Total Weight: 6,731			

WIRE CARRIERS ARE THE PROPERTY OF INSTEEL WIRE PRODUCTS

TO RETURN PLEASE CALL 1-336-719-9000

Subject to section 7 of conditions of applicable bill of lading, if this shipment is to be delivered to the consignee without recourse to the consignor, the consignor shall sign the following statement:
The carrier shall not make delivery of this shipment without payment of freight and all other lawful charges.

Per _____
Received \$ _____
to apply in prepayment of the charges on the property described hereon

Agent or Cashier
Per _____
(The signature here acknowledges only the amount prepaid)

Charges Advanced
\$ _____

BY
TRUCK
FREIGHT
(SCAC) _____

C.O.D. SHIPMENT
C.O.D. Amt _____
Collection Fee _____
Total Charges _____

If charges are to be prepaid, write or stamp here: "To be prepaid"

Prepaid

SPECIAL INSTRUCTIONS

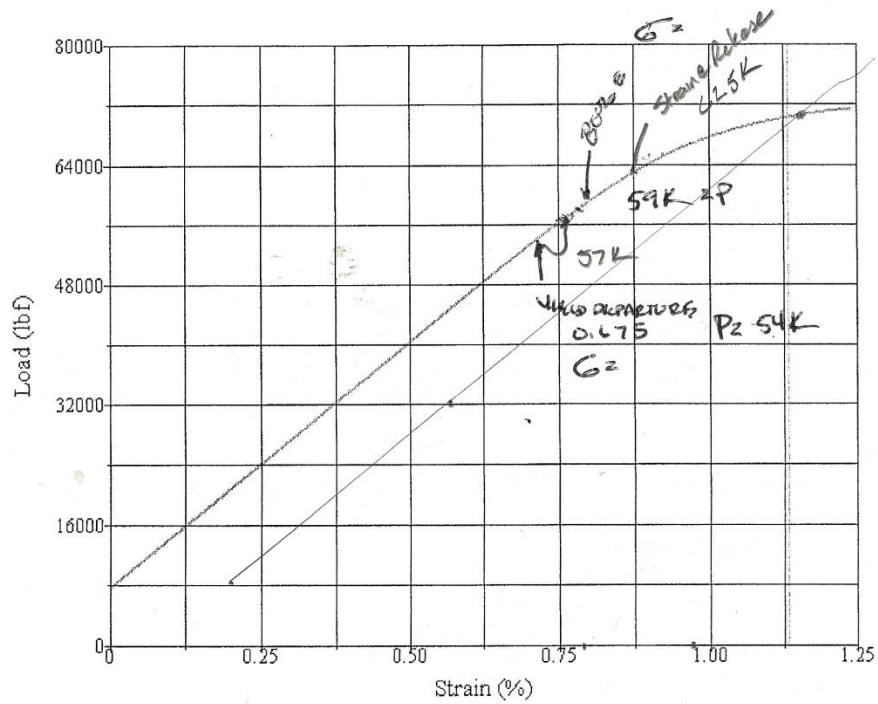
POINT OF DELIVERY - CUST. SIGN.

TIME IN: TIME OUT:

NOTE - Where the rate is dependent on value, shippers are required to state specifically in writing the agreed to declared value of the property.
The agreed or declared value of the property is hereby specifically stated by the shipper to be not exceeding _____

Shipper Certification	Carrier Certification
INSTEEL WIRE PRODUCTS 1373 BOGGS DRIVE MT. AIRY, NC 27030 This is to certify that the above named materials are properly classified, described, packaged, marked and labeled, and are in proper condition for transportation according to the applicable regulations of the DOT Per _____ Date _____	Carrier acknowledges receipt of packages and required placards. Carrier certifies emergency response information was made available and/or carrier has DOT emergency response guidebook or equivalent document in vehicle Per _____ Date _____ Package Nos _____

0.62" STAND

**Test Summary**

Counter: 47955
 Elapsed Time: 00:02:30
 Procedure Name: SWPC CI 0.62 x 330k
 Start Date: 7/13/2012
 Start Time: 1:45:29 PM
 End Date: 7/13/2012
 End Time: 1:47:59 PM
 Workstation: DCCDR5D1
 Tested By: Leo
 Pack Number: S390080A
 3

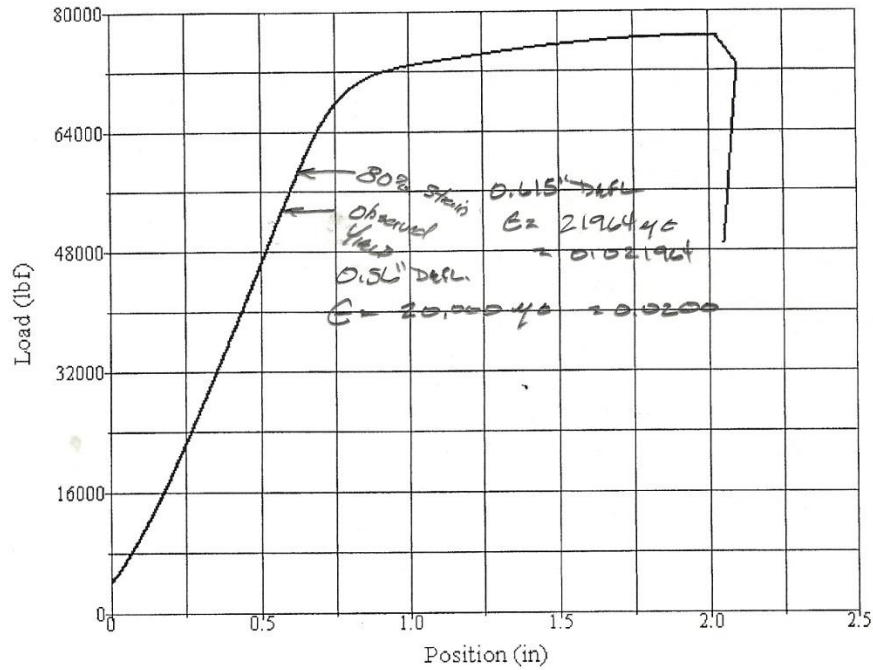
Test Results

Area: 0.2325 in²
 Load at EUL: 70163 lbf
 Young's Modulus: 28.08 Msi

Strain Release
 = 0.00811749 calcd.
 Load from Chart

1.14%

$$\frac{125}{20} = \frac{12}{x}$$



Test Summary

Counter: 48061
 Elapsed Time: 00:02:16
 Procedure Name: Sumiden BS All
 Start Date: 7/16/2012
 Start Time: 12:38:38 PM
 End Date: 7/16/2012
 End Time: 12:40:54 PM
 Workstation: DCCDR5D1
 Tested By: Leo
 Pack Number: SM390030A

Test Results

Area: 0.2425 in²
 Maximum Elongation: 7.4900 % = 2.092"
 Peak Load: 76551.0000 lbf
 Stress at Break: 3.00e+005 psi
 Specimen Gage Length: 28.0000 in



Houston North 6000 Jensen Drive Houston, TX 77026 Tel: 713-697-7105 Fax: 713-697-5945 TF: 800-231-1034	Houston South 8603 Monroe Road Houston, TX 77061 Tel: 713-941-1941 Fax: 713-941-1499 TF: 800-231-1034	San Antonio 2042 W. Thompson Place San Antonio, TX 78226 Tel: 210-431-0088 Fax: 210-431-0701 TF: 800-725-4776	Knoxville 4800 Beverly Road Knoxville, TN 37918 Tel: 865-687-1251 Fax: 865-687-0992 TF: 800-251-9572	Salt Lake City 1840 South 700 West Salt Lake City, UT 84104 Tel: 801-433-2211 Fax: 801-433-2213 TF: 855-433-2211
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www.sss-steel.com

8-29-13
Bed #1

P - 60,600 - 5K

A13-35-1

L - 222' or 2664"

In steel wire

A - .2923

.700 270K

E - 29.0"

Reel #s 1303462109036

1303462109037

Test # 10073690

PL = $\frac{55,600 \times 2664"}{148,1184"} = 17.47"$

AE .2923 $\times$ 29.0" = 8.4767"

$\frac{1}{8}"$ Dead end seating = 0"

$\frac{3}{4}"$ Live end seating = 2387"

$\frac{.75}{17.47} \times 55,600 = 2387" + 60,600 = 62,987"$

Gross elongation = $17\frac{1}{2}" + \frac{3}{4}" + \frac{1}{8}" = 18\frac{3}{8}" @ 62,987"$

Net elongation = $17\frac{1}{2}" + \frac{1}{8}" = 17\frac{5}{8}" @ 60,600"$

$17\frac{5}{8}" + 5\% = 18\frac{1}{2}"$

$17\frac{5}{8}" - 5\% = 16\frac{3}{4}"$

$62,987" \times 5\% = 3149"$

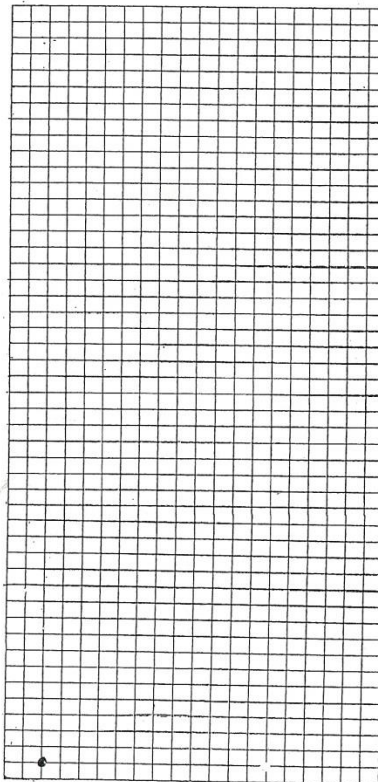
18 $\frac{1}{4}"$



TENSION REPORT

County: KNOX TN
 Job No: A13-35-1
 Dress Date: 8-30-13
 Pour Date: _____
 Piece Marks: _____
 Plant: KNOXville
 Stress Jack: #3

Bed Length: 222' or 2664" Page _____ of _____
 Strain: 60,600 @ 17 1/2 + 1/2 = 17 3/8"
 Preload: 5K
 Live End Seating: 3/4" @ 2387#
 Strand Mfg: IN steel Wire Products
 Size: .700 Type: 270K
 M.O.E. 29.06 Area: .2923
 Reel Numbers: 1303462109036

STRAND LOCATION		ELONGATION		GAUGE		5% ALLOWABLE DEV.		STRAND LOCATION
Horiz. Row	Horiz. Loc.	17 5/8" Calc.	Observ.	" Calc.	Observ.	Elong.	Gauge	
A	1	18 3/4 - 3/4	18 1/4	63,000	63,000	.88"	3149 #	
A	2					16 3/4 - 18 1/2"		

A

0.7

Bed Number: 1

West A

Start Time: _____ Finish Time: _____ By: DAA
 Bed Number: 1 West End



Houston North 6000 Jensen Drive Houston, TX 77026 Tel: 713-697-7105 Fax: 713-697-5945 TF: 800-231-1034	Houston South 8603 Monroe Road Houston, TX 77061 Tel: 713-941-1941 Fax: 713-941-1499 TF: 800-231-1034	San Antonio 2042 W. Thompson Place San Antonio, TX 78226 Tel: 210-431-0088 Fax: 210-431-0701 TF: 800-725-4776	Knoxville 4800 Beverly Road Knoxville, TN 37918 Tel: 865-687-1251 Fax: 865-687-0992 TF: 800-251-9572	Salt Lake City 1840 South 700 West Salt Lake City, UT 84104 Tel: 801-433-2211 Fax: 801-433-2213 TF: 855-433-2211
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www.sss-steel.com

8-30-13

.62 330K

P - 58,100 - 5K

L - 222'

A - .2348

E - 28.5

$$\frac{PL}{AE} = \frac{141,458,400^6}{6,691,800^6} = 21.14''$$

$$\frac{.625}{21.14} \times 58,100 = 1570^{\#} + 58100 = 59,670$$

5/8" Live end seating = 1570[#]

1/8" Dead end seating = 0[#]

$$\text{Gross elongation} = 21\frac{1}{8} + \frac{1}{8} + \frac{5}{8} = 21\frac{3}{8}''$$

$$\text{Net elongation} = 21\frac{1}{8} + \frac{1}{8} = 21\frac{1}{4}'' @ 58,100^{\#}$$

$$21\frac{1}{4}'' + 5\frac{1}{8}'' = 22\frac{3}{8}''$$

$$21\frac{1}{4}'' - 5\% = 20\frac{1}{8}''$$

$$58100 \times 5\% = 2905^{\#}$$

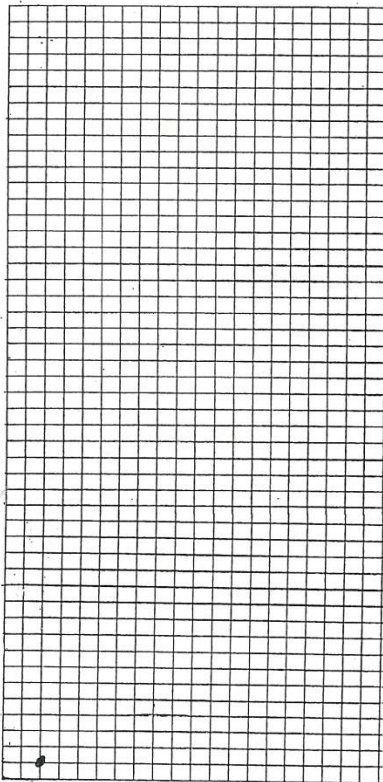
21 5/8"



TENSION REPORT

County: KNOX TN
 PC Job No: A13-35-1 UT
 Stress Date: 8-30-13
 Pour Date: _____
 Piece Marks: _____
 Plant: KNOXVILLE
 Stress Jack: #3

Bed Length: 222' or 2664" Page _____ of _____
 Strain: 58,100 @ 21 1/8" + 1/8" = 21 1/4"
 Preload: 5K
 Live End Seating: 5/8" = 1570# Dead End = 1/8" = 0#
 Strand Mfg: _____
 Size: .62 Type: 330K
 M.O.E. 28.56 Area: .2348
 Reel Numbers: _____

STRAND LOCATION		ELONGATION		GAUGE		5% ALLOWABLE DEV.		STRAND LOCATION
Horiz. Row	Horiz. Loc.	Calc.	Observ.	Calc.	Observ.	Elong.	Gauge	
A	1	21 1/8"	21 3/8"	59700	59700	1.06"	2905	
						20 1/8" - 22 3/8"		

Start Time: _____ Finish Time: _____ By: DAH

Bed Number: 1 West End

Appendix B – Concrete Material Information

Development Length Test ^{A13-35-1}

5000 PSI Mix
Poured 9-03-13

80,000 6366 PSI
80,000 6366 PSI

Poured 9-04-13
7500 PSI Mix

89,000 7082 PSI } 7000
90,000 7162 PSI }

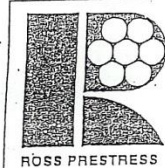
Poured 9-05-13

10,000 Mix

127,000 10,106 PSI } 10,000
127,500 10,146 PSI }

Jeremy 437-7480

Dispatch

**ROSS PRESTRESSED CONCRETE, INC.**

2701 Independence Lane
P.O. Box 6299
Knoxville, Tennessee 37914
Phone (865) 524-1485
Fax (865) 637-0831

ROSS PRESTRESS

PRODUCTION REPORT*Last Load Poured: 6:00PM*COUNTY, ST. KNOX TNRPC JOB NO. A13-35-1POUR NO. 1POUR DATE 9-03-13AIR TEMP. 83°INSPECTORS DAPLANT: KNOXVILLE**CONCRETE PROPERTIES**

LOAD NO. 1 of 1
STATION East Bed 1
SLUMP 6"
AIR CONTENT NA
PLACEMENT TEMP. 85°
COMP. STRENGTH @ DET. 9-9-13 8:00AM 6366, 6366 PSI
COMP. STRENGTH @ 28 DAYS 9-6-13 4775 PSI

MEMBERS CAST: BM-3 & BML6DESCRIPTION: 8"X12" X50' Test Beams**REQUIRED PROPERTIES**

TYPE MIX Class A
COMP. STRENGTH @ DET. 5000 PSI
COMP. STRENGTH @ 28 DAYS 5000 PSI
ADMIXTURES 7700-40oz, 300R-20oz
AIR CONTENT NA

PRE-POUR INSPECTION

	PRE-POUR INSPECTION	DA
STRAND	☒	FORM COND. ☒
RE-STEEL	☒	SKEW <u>NA</u>
HEIGHT	☒	BLOCKOUTS <u>NA</u>
WIDTH	☒	INSERTS <u>NA</u>
LENGTH	☒	PLATES <u>NA</u>
VOIDS	<u>NA</u>	DEFL. PTS. <u>NA</u>
FORM ALIGN.	☒	END DETAILS ☒

POST-POUR INSPECTION

HEIGHT _____
WIDTH _____
LENGTH _____
CAMBER _____
SKEW _____
INSERTS _____
DOWEL HOLES _____
TIE ROD HOLES _____
FINISH _____
MISC. _____

TENSION DATA

MFG. _____
SIZE/TYPE _____
INIT. TENSION _____
PRELOAD _____
BED LENGTH _____
NET ELONG. _____
MAX GAUGE _____
MIN GAUGE _____
STRESS DATE _____
REEL NO. _____
TENN: YES _____ NO _____
MACHINE #3
See Tension Report

AMBIENT CURING TEMP**LOCATIONS & TEMP**

TIME			

REMARKS: Add 1 gal/cy water @ Bed

QUALITY CONTROL SUPERVISOR _____

BATCH REPORT KNOXVILLE PLANT

DATE: 9-3-13
BED #: 1

OWNER: UT
MIX: Barrier

LOAD	C.Y.	CEMENT	FLY ASH	FINE	TOTAL AGG.	METERED WATER	ADDED WATER	RETARDER OR W. RED.	HRWR (SPECIFY)	SILICA	OTHER
1	3.0	1692		5240	10400	51	4	60	120		
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15	124	564	Ø	1744	3460	18.2		2002	4002		

FA 1744 @ 5.99%
CA 1716 @ 0.96%

PULL WEIGHT SUMMARY BASED ON MOISTURE

09/03/13

Mix #: Barriers 5000 psi mix

Free Moisture F.A. = 5.99 %
Free Moisture C.A. = 0.96 %
Total Free Moisture = 13.8 Gal./C.Y.

SSD		
<u>Item</u>	<u>Qty. for 1 CY</u>	
Cement	564	Pounds
Fine Agg.	1645	Pounds
Coarse Agg.	1700	Pounds
Water	32.0	Gallons
300R	20.0	Ounces
Silica Fume	0.0	Pounds
Air Ent.	0.0	Ounces
7700	40	Ounces

<u>Item</u>	<u>1/4 CY</u>	<u>1/2 CY</u>	<u>3/4 CY</u>	<u>1 CY</u>	<u>1-1/4 CY</u>	<u>1-1/2 CY</u>	<u>1-3/4 CY</u>	<u>2 CY</u>
Cement	141	282	423	564	705	846	987	1128
Fine Agg.	436	872	1308	1744	2179	2615	3051	3487
Coarse Agg.	429	858	1287	1716	2145	2574	3004	3433
Total Agg.	865	1730	2595	3460	4325	5190	6055	6920
Water	4.6	9.1	13.7	18.2	22.8	27.3	31.9	36.4
200N	5.0	10.0	15.0	20.0	25.0	30.0	35.0	40.0
Silica Fume	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Air Ent.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
RB								

<u>Item</u>	<u>2-1/4 CY</u>	<u>2-1/2 CY</u>	<u>2-3/4 CY</u>	<u>3 CY</u>	<u>3-1/4 CY</u>	<u>3-1/2 CY</u>	<u>3-3/4 CY</u>	<u>4 CY</u>
Cement	1269	1410	1551	1692	1833	1974	2115	2256
Fine Agg.	3923	4359	4795	5231	5666	6102	6538	6974
Coarse Agg.	3862	4291	4720	5149	5578	6007	6436	6865
Total Agg.	7785	8650	9515	10380	11245	12109	12974	13839
Water	41.0	45.5	50.1	54.7	59.2	63.8	68.3	72.9
200N	45.0	50.0	55.0	60.0	65.0	70.0	75.0	80.0
Silica Fume	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Air Ent.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
RB								

BATCH REPORT KNOXVILLE PLANT

DATE: 9-4-13
BED #: 1

OWNER: A13-35-1
MIX: ATN-05F w/TPJ
Cement

LOAD	C.Y.	CEMENT	FLY ASH	FINE	TOTAL AGG.	METERED WATER	ADDED WATER	RETARDER OR W. RED.	HRWR (SPECIFY)	SILICA	OTHER
1	3.0	1584	398	4770	10210	60		60	150		3.42
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15	104	526	132	1590	3397	21.8		2002	5002		

FA 1590 @ 4.31 90
CA 1808 @ 1.55 90

PULL WEIGHT SUMMARY BASED ON MOISTURE

09/04/13

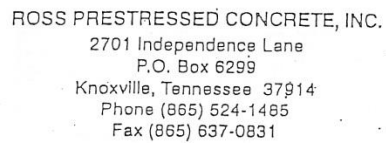
Mix #: KTN-05 With Flyash
658 w/ 20% Replacement

Free Moisture F.A. = 4.31 %
Free Moisture C.A. = 1.55 %
Total Free Moisture = 11.2 Gal./C.Y.

<u>Item</u>	<u>SSD</u> <u>Qty. for 1 CY</u>	<u>Type III</u>
Cement	526 Pounds	<i>Type I</i>
Flyash	132 Pounds	
Fine Agg.	1524 Pounds	
Coarse Agg.	1780 Pounds	
Water	33.0 Gallons	
300R	20.0 Ounces	
7700	53.0 Ounces 48.0	

<u>Item</u>	<u>1/4 CY</u>	<u>1/2 CY</u>	<u>3/4 CY</u>	<u>1 CY</u>	<u>1-1/4 CY</u>	<u>1-1/2 CY</u>	<u>1-3/4 CY</u>	<u>2 CY</u>
Cement	132	263	395	526	658	789	921	1052
Flyash	33	66	99	132	165	198	231	264
Fine Agg.	397	795	1192	1590	1987	2385	2782	3179
Coarse Agg.	452	904	1356	1808	2259	2711	3163	3615
Total Agg.	849	1699	2548	3397	4247	5096	5945	6795
Water	5.5	10.9	16.4	21.8	27.3	32.7	38.2	43.6
300R	5.0	10.0	15.0	20.0	25.0	30.0	35.0	40.0
7700	13.3	26.5	39.8	53.0	66.3	79.5	92.8	106.0

<u>Item</u>	<u>2-1/4 CY</u>	<u>2-1/2 CY</u>	<u>2-3/4 CY</u>	<u>3 CY</u>	<u>3-1/4 CY</u>	<u>3-1/2 CY</u>	<u>3-3/4 CY</u>	<u>4 CY</u>
Cement	1184	1315	1447	1578	1710	1841	1973	2104
Flyash	297	330	363	396	429	462	495	528
Fine Agg.	3577	3974	4372	4769	5166	5564	5961	6359
Coarse Agg.	4067	4519	4971	5423	5875	6327	6778	7230
Total Agg.	7644	8493	9343	10192	11041	11890	12740	13589
Water	49.1	54.5	60.0	65.4	70.9	76.3	81.8	87.2
300R	45.0	50.0	55.0	60.0	65.0	70.0	75.0	80.0
7700	119.3	132.5	145.8	159.0	172.3	185.5	198.8	212.0



PRODUCTION REPORT

Last Load Poured: 4:38 PM

REQUIRED PROPERTIES

TYPE MIX	<u>KTN-01F</u>
COMP. STRENGTH @ DET.	
COMP. STRENGTH @ 28 DAYS	<u>10,000 PSI</u>
ADMIXTURES	<u>7700-6.3oz, 300R-21oz</u>
AIR CONTENT	<u>NA</u>

PRE-POUR INSPECTION *DA*

STRAND	✓	FORM COND.	✓
RE-STEEL	✓	SKEW	NA
HEIGHT	✓	BLOCKOUTS	NA
WIDTH	✓	INSERTS	NA
LENGTH	✓	PLATES	NA
VOIDS	NA	DEFL. PTS.	NA
FORM ALIGN.	✓	END DETAILS	✓

AMBIENT CURING TEMP

MFG. _____
 SIZE/TYPE _____
 INIT. TENSION _____
 PRELOAD _____
 BED LENGTH _____
 NET ELONG. _____
 MAX GAUGE _____
 MIN GAUGE _____
 STRESS DATE _____
 REEL NO. _____
 TENN: YES _____ NO _____
 MACHINE _____ # 3 _____
 See Tension Report

[illegible]

REMARKS: BM1 - .62 BM4 - .7

QUALITY CONTROL SUPERVISOR

BATCH REPORT KNOXVILLE PLANT

DATE: 9-5-13
BED #: 1

OWNER: A13-35-1
MIX: HTN-01F

LOAD	C.Y.	CEMENT	FLY ASH	FINE	TOTAL AGG.	METERED WATER	ADDED WATER	RETARDER OR W. RED.	HRWR (SPECIFY)	SILICA	OTHER
1	3.0	1696	428	4580	9990	70		63	190		3.18
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15	10.4	564	141	1525	3329	25		21 02	6302		

FA 1525 @ 2.8990
CA 1804 @ 1.3590

PULL WEIGHT SUMMARY BASED ON MOISTURE

09/05/13

Mix #: KTN-01 With Flyash
705 w/ 20% Replacement

Free Moisture F.A. = 2.89 %
Free Moisture C.A. = 1.35 %
Total Free Moisture = 8.0 Gal./C.Y.

<u>Item</u>	<u>SSD</u> <u>Qty. for 1 CY</u>	
Cement	564	Pounds
Flyash	141	Pounds
Fine Agg.	1482	Pounds
Coarse Agg.	1780	Pounds
Water	33.0	Gallons
300R	21.0	Ounces
7700	67.0	Ounces

1000000

<u>Item</u>	<u>1/4 CY</u>	<u>1/2 CY</u>	<u>3/4 CY</u>	<u>1 CY</u>	<u>1-1/4 CY</u>	<u>1-1/2 CY</u>	<u>1-3/4 CY</u>	<u>2 CY</u>
Cement	141	282	423	564	705	846	987	1128
Flyash	35	71	106	141	176	212	247	282
Fine Agg.	381	762	1144	1525	1906	2287	2668	3050
Coarse Agg.	451	902	1353	1804	2255	2706	3157	3608
Total Agg.	832	1664	2497	3329	4161	4993	5826	6658
Water	6.2	12.5	18.7	25.0	31.2	37.5	43.7	50.0
300R	5.3	10.5	15.8	21.0	26.3	31.5	36.8	42.0
7700	16.8	33.5	50.3	67.0	83.8	100.5	117.3	134.0

<u>Item</u>	<u>2-1/4 CY</u>	<u>2-1/2 CY</u>	<u>2-3/4 CY</u>	<u>3 CY</u>	<u>3-1/4 CY</u>	<u>3-1/2 CY</u>	<u>3-3/4 CY</u>	<u>4 CY</u>
Cement	1269	1410	1551	1692	1833	1974	2115	2256
Flyash	317	353	388	423	458	494	529	564
Fine Agg.	3431	3812	4193	4574	4956	5337	5718	6099
Coarse Agg.	4059	4510	4961	5412	5863	6314	6765	7216
Total Agg.	7490	8322	9154	9987	10819	11651	12483	13315
Water	56.2	62.4	68.7	74.9	81.2	87.4	93.7	99.9
300R	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0
7700	150.8	167.5	184.3	201.0	217.8	234.5	251.3	268.0



AGGREGATES USA

Basic Quality Statistical Summary Report

Period 08/01/2013 - 08/31/2013
Plant Midway Quarry-502
Product # 7 Stone Washed (1/2 TO #4)-7000
Specification #7 Stone

Sieve/Test	Tests	Average	St Dev	Target	Specification
3/4" (19mm)	9	100.0	0.00		100-100
1/2" (12.5mm)	9	92.4	1.90		90-100
3/8" (9.5mm)	9	58.1	5.78		40-70
#4 (4.75mm)	9	4.2	0.98		0-15
#8 (2.36mm)	9	1.3	0.14		0-5
Pan	9	0.00	0.000		



AGGREGATES USA

Basic Quality Statistical Summary Report

Period 08/01/2013 - 08/31/2013
Plant Midway Quarry-502
Product #67 Stone Washed (3/4 TO #4)-6700
Specification #67 Stone

Sieve/Test	Tests	Average	St Dev	Target	Specification
1" (25mm)	1	100.0			100-100
3/4" (19mm)	1	95.2			90-100
1/2" (12.5mm)	1	52.0			
3/8" (9.5mm)	1	31.6			20-55
#4 (4.75mm)	1	4.4			0-10
#8 (2.36mm)	1	0.4			0-5
Pan	1	0.00			



AGGREGATES USA

Basic Quality Statistical Summary Report

Period 08/01/2013 - 08/31/2013
Plant Midway Quarry-502
Product Mfg Concrete Sand Limestone (Washed Process A)-9300
Specification Mfg Concrete Sand Limestone

Sieve/Test	Tests	Average	St Dev	Target	Specification
3/8" (9.5mm)	21	100.0	0.00	100-100\100	100-100
#4 (4.75mm)	21	100.0	0.00	100-100\100	95-100
#8 (2.36mm)	21	92.6	0.67	90.2-94.5\92.3	
#10 (2mm)	21	81.8	1.01	77.9-85.1\81.5	
#16 (1.18mm)	21	53.6	1.64	50-58.5\53.3	50-90
#30 (0.6mm)	21	32.0	1.76	26.5-38\32.2	
#50 (0.3mm)	21	17.7	1.57	13.1-23.2\18.1	5-30
#100 (0.15mm)	21	7.8	1.11	5.2-10\8	0-10
#200 (75um)	21	3.58	0.629	2.45-4.93\3.69	0-5
Pan	20	0.00	0.000		
FM	21	2.96	0.062	2.8-3\2.9	2.3-3.1

Comments

Query
Query Selections
Date Created 09/05/2013
Date Range 08/01/2013 - 08/31/2013
Plant Midway Quarry
Sample Type Shipping
Sample Method Stockpile



Client: Mr. Tom Hendrix
The SEFA Group
P.O. Box 6
Moncks Corner, SC 29461

Date: June 26, 2012
TEC Services I.D.: TEC 06-0509
Lab No.: 12-201-CUFP

REPORT OF FLY ASH TESTS				
Sample I.D. No.: CFP043012		Date Sampled: April 30, 2012		
Manufacturer: Cumberland Fossil Plant		Date Received: May 15, 2012		
Chemical Analysis**	Results	Specification (Class F)		
		ASTM C618-08	AASHTO M295-07	
Silicon Dioxide	51.25	----	----	
Aluminum Oxide	20.16	---	---	
Iron Oxide	14.81	----	----	
Sum of Silicon Dioxide, Iron Oxide & Aluminum Oxide	86.22	70 % min.	70 % min.	
Calcium Oxide	5.80	----	----	
Magnesium Oxide	1.09	----	----	
Sulfur Trioxide	1.58	5 % max.	5 % max.	
Loss on Ignition	1.11	6 % max.	5 % max.	
Moisture Content	0.07	3 % max.	3 % max.	
Available Alkalies as Na ₂ O	0.61	----	1.5 % max.	
Physical Analysis				
Fineness (Amount Retained on #325 Sieve)		16.4%	34 % max.	34 % max.
Strength Activity Index with Portland Cement				
At 7 Days:				
Control Average, psi: 4580	Test Average, psi: 3460	76%	75 % min. (of control)	75 % min. (of control)
At 28 Days:				
Control Average, psi: 5600	Test Average, psi: 4450	79%	75 % min. (of control)	75 % min. (of control)
Water Requirements (Test H ₂ O/Control H ₂ O)				
Control, mls: 242	Test, mls: 237	98%	105 % max. (of control)	105 % max. (of control)
Autoclave Expansion		0.02%	± 0.8 % max.	± 0.8 % max.
Uniformity Requirements				
Specific Gravity: 2.42	Average: 2.42	0.1%	5 % max. from average	5 % max. from average
% Retained #325 Sieve: 16.4	Average: 15.5	0.9%	5 % max. from average	5 % max. from average

† Meeting the 7 day or 28 day strength activity index will indicate specification compliance

• Optional

**Chemical Analysis performed by Wyoming Analytical

The results of our testing indicate that this sample complies with ASTM C618-08a and AASHTO M295-07 specifications for Class F pozzolan

Respectfully Submitted,

Testing, Engineering & Consulting Services, Inc.

Anne Miller

Shawn McCormick

Testing, Engineering & Consulting Services, Inc.
235 Buford Drive | Lawrenceville, GA 30046
770.995.8000 | 770.995.8550 (fl) | www.tecservices.com



Buzzi Unicem USA

Chattanooga Plant
P.O. Box 4304
1201 Suck Creek Road
Chattanooga, TN 37405
Phone: (423) 886-0800
Fax: (423) 886-4651

Mill Test Report

Cement Type: Type I/H Low Alkali
Manufacture: July, 2013
Silo Number: _____

Chemical

SiO ₂ (%)	19.5
Al ₂ O ₃ (%)	4.6
Fe ₂ O ₃ (%)	3.6
CaO (%)	63.3
MgO (%)	3.6
SO ₃ (%)	2.7
Total Alkali (Na ₂ O + 0.658K ₂ O)	0.56
Ignition Loss	2.0
Insoluble Residue	0.35
C ₃ S (%)	65.8
C ₂ S (%)	6.2
C ₃ A (%)	6.1
C ₄ AF (%)	10.8
C ₃ S + 4.75C ₃ A	94.8
CO ₂ (%)	1.5
Limestone (%)	3.8
CaCO ₃ in Limestone (%)	90.5

Physical

Time of Set (Vicat)		
Initial Set (min.)	116	
Final Set (min.)	237	
Compressive Strength		
	PSI	MPa
1 Day	2296	15.8
3 Day	3817	26.3
7 Day	4780	33.0
28 Day	6522	45.0
Cube Flow		
	108	
Fineness, Blaine (cm ² /g)		
	3720	
325 Mesh (%)		
	96.4	
Air Content (%)		
	6.8	
Normal Consistency (%)		
	24.6	
False Set (%)		
	66	
Autoclave Expansion (%)		
	0.03	

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirements of ASTM C-150, AASHTO M-85, or ASTM C-91.

received 8.28-13

1/2 load

File B

KEEL

R. J. Delashmitt

Randall Delashmitt, Quality Control Manager

Date 8/8/2013 10:23:30 AM



Buzzi Unicem USA

Chattanooga Plant
P.O. Box 4304
1201 Suck Creek Road
Chattanooga, TN 37405
Phone: (423) 886-0800
Fax: (423) 886-4651

Mill Test Report

Cement Type: Type III Low Alkali
Manufacture: June, 2013
Silo Number: _____

Chemical

SiO ₂ (%)	<u>20.2</u>
Al ₂ O ₃ (%)	<u>4.6</u>
Fe ₂ O ₃ (%)	<u>3.5</u>
CaO (%)	<u>63.1</u>
MgO (%)	<u>3.7</u>
SO ₃ (%)	<u>2.8</u>
Total Alkali (Na ₂ O + 0.658K ₂ O)	<u>0.57</u>
Ignition Loss	<u>2.2</u>
Insoluble Residue	<u>0.29</u>
C ₃ S (%)	<u>59.9</u>
C ₂ S (%)	<u>12.7</u>
C ₃ A (%)	<u>6.2</u>
C ₄ AF (%)	<u>10.6</u>
C ₃ S + 4.75C ₃ A	<u>89.4</u>
CO ₂ (%)	<u>1.5</u>
Limestone (%)	<u>3.7</u>
CaCO ₃ in Limestone (%)	<u>91.0</u>

Physical

Time of Set (Vicat)		
Initial Set (min.)	<u>85</u>	
Final Set (min.)	<u>185</u>	
Compressive Strength	PSI	MPa
1 Day	<u>3530</u>	<u>24.3</u>
3 Day	<u>5290</u>	<u>36.5</u>
7 Day	<u>6380</u>	<u>44.0</u>
28 Day	<u>-</u>	<u>-</u>
Cube Flow	<u>98</u>	
Fineness, Blaine (cm ² /g)	<u>5970</u>	
325 Mesh (%)	<u>100.0</u>	
Air Content (%)	<u>8.0</u>	
Normal Consistency (%)	<u>29.2</u>	
False Set (%)	<u>62</u>	
Autoclave Expansion (%)	<u>0.05</u>	

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirements of ASTM C-150, AASHTO M-85, or ASTM C-91.

Received 8-23-13
52350 #
Silo A
REL

R. J. Delashmitt

Randall Delashmitt, Quality Control Manager

Date 7/8/2013 1:00:51 PM



Description

Glenium 7700 ready-to-use high-range water-reducing admixture is based on the next generation of polycarboxylate technology found in all of the Glenium 7000 series products. This technology combines state-of-the-art molecular design with a precise understanding of regional cements to provide specific and exceptional value to all phases of the concrete construction process.

Glenium 7700 is particularly effective in improving the day to day consistency of concrete mixtures. This is accomplished by providing unparalleled slump retention without compromising early compressive strength development and setting time. Workability retention helps to maintain more consistent performance from batch to batch, thereby improving operational efficiencies. Glenium 7700 meets ASTM C 494 provisional compliance requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications

Recommended for use in:

- Concrete requiring high-early compressive strength development
- Applications requiring workability retention without retardation
- Concrete where high flowability, increased stability and durability are needed
- Production of Self-Consolidating Concrete (SCC) mixtures
- Concrete with varying water reduction requirements (5-40%)

Glenium® 7700

High-Range *Water Reducing Admixture*

Features

- Superior slump retention
- Excellent early strength development
- High ultimate strengths
- Optimum setting time
- Consistent air entrainment
- Dosage flexibility

Benefits

- Consistency in placement operations
- Optimized mixture costs
- Reduction in patching costs
- Ability to attain difficult combinations of high-early and late-age compressive strengths
- Increased productivity
- Improved operational efficiencies
- Less QC support
- Fewer rejected loads
- Faster form turnover
- Workability retention without retardation

Performance Characteristics

Slump Retention: Concrete produced with Glenium 7700 admixture maintains slump significantly longer than concrete mixtures containing naphthalene, melamine, and first generation polycarboxylate high-range water-reducing admixtures. This slump retention is achieved without affecting rate of hardening or early age compressive strength development.

Mixture Data: Figure 1 represents average slump retention performance across multiple field trials throughout North America. Figure 2 represents average compressive strength results from these same field trials. Materials were all different as were starting consistencies. The reference admixture in this graph represents the first generation of high-early strength polycarboxylate technology.

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Builders

Product Data: Glenium 7700

Figure 1. Average Slump Retention Performance

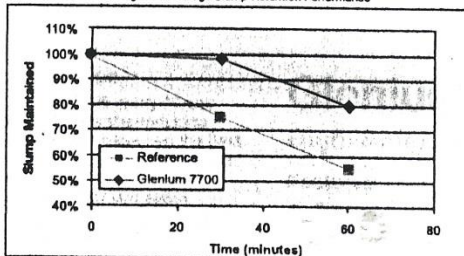
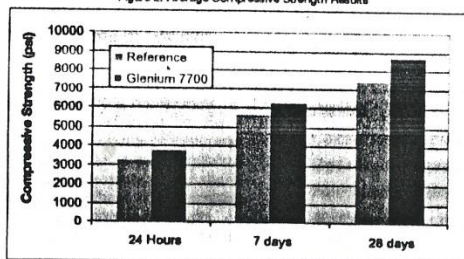


Figure 2. Average Compressive Strength Results



Guidelines for Use

Dosage: Glenium 7700 has a recommended dosage range of 4-15 fl oz/cwt (260-975 mL/100 kg) of cementitious materials. For most applications, dosages in the range of 4-12 fl oz/cwt (260-780 mL/100 kg) will provide excellent performance. For very high performance and Rheodynamic® Self-Consolidating concrete mixtures, up to 15 fl oz/cwt (975 mL/100 kg) of cementitious materials can be utilized. Because of variations in concrete materials, job site conditions and/or applications, dosages outside of the recommended range may be required. In such cases, contact your local sales representative.

Mixing: Glenium 7700 admixture can be added with the initial batch water or as a delayed addition. However, optimum water reduction is generally obtained with a delayed addition.

Product Notes

Corrosivity - Non-Chloride, Non-Corrosive: Glenium 7700 admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of Glenium 7700 admixture.

Compatibility: Glenium 7700 admixture is compatible with most admixtures used in the production of quality concrete, including normal, mid-range and high-range water-reducing admixtures, accelerators, retarders, extended set control admixtures, air-entrainers, corrosion inhibitors, and shrinkage reducers.

Do not use Glenium 7700 admixture with admixtures containing beta-naphthalene sulfonate. Erratic behaviors in slump, workability retention and pumpability may be experienced.

Storage and Handling

Storage Temperature: Glenium 7700 admixture must be stored at temperatures above 40 °F (5 °C). If Glenium 7700 admixture freezes, thaw and reconstitute by mechanical agitation. Do not use pressurized air for agitation without consulting technical support.

Shelf Life: Glenium 7700 admixture has a minimum shelf life of 6 months. Depending on storage conditions, the shelf life may be greater than stated. To ensure the longest shelf life potential, recirculation is recommended. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of Glenium 7700 admixture has been exceeded.

Packaging

Glenium 7700 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: Glenium 7700 admixture

Additional Information

For additional information on Glenium 7700 admixture or its use in developing concrete mixtures with special performance characteristics, contact your local sales representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative admixtures for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

BASF Construction Chemicals, LLC
Admixture Systems

www.masterbuilders.com

United States: 23700 Chagrin Boulevard, Cleveland, Ohio 44122-5544 • Tel: 800 628-9900 • Fax: 216 639-8821
Canada: 1800 Clark Boulevard, Brampton, Ontario L6T 4M7 • Tel: 800 387-5982 • Fax: 905 792-0651

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BASF

The Chemical Company

Description

Pozzolith 300 R ready-to-use, liquid admixture is used for making more uniform and predictable high-performance concrete while retarding its setting time to facilitate placing and finishing operations.

It meets ASTM C 494/C 494M requirements for Type B, retarding, and Type D, water-reducing and retarding, admixtures.

Applications

Recommended for use in:

- Prestressed concrete
- Precast concrete
- Reinforced concrete
- Shotcrete
- Lightweight or normal weight concrete
- Pumped concrete
- 4x4™ Concrete
- Pervious Concrete
- Rheodynamic® Self-Consolidating Concrete (SCC)

POZZOLITH® 300 R

Set Retarding Admixture

Features

- ☛ Reduced water content required for a given workability
- ☛ Retarded setting characteristics
- ☛ Improved workability
- ☛ Reduced segregation
- ☛ Superior finishing characteristics for flatwork and cast surfaces
- ☛ Moderate to extended retardation – depending on the dosage rate
- ☛ Dead-load deflection can take place (before concrete sets) in extended pours for bridge decks, cantilevers, nonshored structural elements, etc.

Benefits

- ☛ Flexibility in the scheduling of placing and finishing operations
- ☛ Offsets the effects of early hardening during extended delays between mixing and placing
- ☛ Helps eliminate cold joints
- ☛ Peak temperature and/or rate of temperature rise in mass concrete lowered thereby reducing thermal cracking
- ☛ Increased compressive and flexural strength

Performance Characteristics

Concrete produced with Pozzolith 300 R admixture will have rapid strength development after initial set occurs. It develops higher early (24-hour) and ultimate strengths than plain concrete when used within the recommended dosage range, under normal, comparable curing conditions.

When Pozzolith 300 R admixture is used in heat-cured concrete, the length of the preheating period should be increased until initial set of the concrete is achieved. The actual heat-curing period is then reduced accordingly to maintain existing production cycles without sacrificing early or ultimate strengths.

Rate of Hardening: The temperature of the concrete mixture and the ambient temperature (forms, earth, reinforcement, air, etc.) affect the hardening rate of concrete. At higher temperatures, concrete hardens more rapidly which may cause problems with placing and finishing. Pozzolith 300 R admixture retards the set of concrete. Within the normal dosage range, it will generally extend the working and setting times of concrete containing normal portland cement approximately 1 hour to 5 hours compared to that of a plain concrete mixture, depending on job materials and temperatures.

Trial mixtures should be made with job materials approximating job conditions to determine the dosage required.

Master
Builders

Product Data: POZZOLITH 300 R

Guidelines for Use

Dosage: Pozzolith 300 R admixture is recommended for use at a dosage of 4 ± 1 fl oz/cwt (260 ± 65 mL/100 kg) of cementitious materials for most concrete mixtures using typical concrete ingredients. Because of variations in job conditions and concrete materials, dosages other than the recommended amounts may be required. In such cases, contact your BASF Construction Chemicals representative.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: Pozzolith 300 R admixture will neither initiate nor promote corrosion of reinforcing steel in concrete. This admixture does not contain intentionally-added calcium chloride or other chloride-based ingredients.

Compatibility: Pozzolith 300 R admixture may be used in combination with any BASF Construction Chemicals admixtures. When used in conjunction with other admixtures, each admixture must be dispensed separately into the mix.

Storage and Handling

Storage Temperature: If Pozzolith 300 R admixture freezes, thaw at 35°F (2°C) or above and completely reconstitute by mild mechanical agitation. **Do not use pressurized air for agitation.**

Shelf Life: Pozzolith 300 R admixture has a minimum shelf life of 18 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of Pozzolith 300R admixture has been exceeded.

Packaging

Pozzolith 300 R admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: Pozzolith 300 R admixture.

Additional Information

For additional information on Pozzolith 300 R admixture or its use in developing a concrete mixture with special performance characteristics, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

BASF Construction Chemicals, LLC Admixture Systems

www.masterbuilders.com

United States 23700 Chagrin Boulevard, Cleveland, Ohio 44122-5544 Tel: 800 628-9990 Fax: 216 839-8821
Canada 1800 Clark Boulevard, Brampton, Ontario L6T 4M7 Tel: 800 387-5862 Fax: 905 792-0651

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Appendix C – Small-Scale Test Data Sheets

Concrete Surface Strain

Specimen: A0.70-6 (Specimen B7M6 West)
 Date Tested: 10/22/13

10/24/13

Load	Initial	Release	4.00 kip	6.50 kip	7.0 kip
1 to 3	0.13588	0.12611	0.12788	0.12822	0.12485
2 to 4	0.21095	—	—	—	—
3 to 5	0.42450	—	—	—	—
4 to 6	0.14764	—	—	—	—
5 to 7	0.16133	0.15570	0.15195	0.15037 ?	0.15138
7 to 8	0.06744	0.08089	0.07511	0.07773	0.07693
8 to 9	—	—	—	—	—
9 to 10	0.27894	0.28762	0.28010	0.28098	0.27923
10 to 11	0.24905	0.26762	0.26465	0.26700 ?	0.26270
11 to 12	0.12943	0.10950	0.10650	0.10469	0.10373
12 to 13	0.07458	0.06570	0.06142	0.05822	0.05792
13 to 14	0.39185	0.32524	0.32857	0.31993	0.31410
14 to 15	0.29339	0.29112	0.28475	0.28714 ?	0.28656
15 to 16	0.41542	0.40544	0.40030	0.38651	0.37251
16 to 17	0.07779	0.08680	0.07646	0.03202	— < 0.00000
17 to 18	0.45942	0.46198	0.46004 ?	0.46	—
18 to 19	0.36000	0.34086	0.33429	0.31188	0.29924
19 to 20	0.27768	0.26042	0.25742	0.23664	0.22240
20 to 21	0.15462	0.15190	0.14928	0.12736	0.10825 ?
21 to 22	0.27346	0.24141	0.25732	0.23713	0.23024135
22 to 23	0.41449	0.41491	0.41216	0.39092	0.38393
23 to 24	0.13605	0.09147	0.08737	0.06092	0.05870
24 to 25	0.18142	0.21150	0.21468 ?	0.21322	0.21798 ?

Concrete Surface Strain

Specimen: Δ0.70-6
 Date Tested: _____

Load	7.5 kips				
1 to 3	0.12988				
2 to 4	—				
3 to 5	—				
4 to 6	—				
5 to 7	0.15285				
7 to 8	0.07691				
8 to 9	—				
9 to 10	0.27980				
10 to 11	0.26841				
11 to 12	0.10814 ^{604?}				
12 to 13	0.06542				
13 to 14	0.29746				
14 to 15	0.28669				
15 to 16	0.34202				
16 to 17	—				
17 to 18	—				
18 to 19	0.24312				
19 to 20	0.18108				
20 to 21	0.07007				
21 to 22	0.21305				
22 to 23	0.36073				
23 to 24	0.04394				
24 to 25	0.21345				

Miscellaneous Testing Data

Specimen: A0.70-6 (Specimen BNL West)
 Date Tested: 10/22/13

Load	INITIAL	Remeasure at West Support 1 kip	2.00 kip	4.00 kip	6.5 kip
Measured Defl 1/4 Span	38 1/16	37 7/8	37.5	37.0	33 15/16
Measured Defl 1/2 Span	38 7/16	38 3/16	37 13/16	37 3/16	34 3/8
Measured Defl 3/4 Span	38 1/4	38 1/4	37 13/16	37 8/16	36.0
Defl @ West End	14 1/16	14 1/16	13 15/16	13 15/16 13 9/16	14 1/32 13 3/8
Defl @ East End		13 1/8	13 7/8	14 1/16 13 15/16	13 31/32 14.0
Bearing Contact West End	Full	Full	Full	Full	Full
Bearing Contact East End	Full	Full	Full	Full	Full
Measured Rotat West End	0	0	0	0	0
Measured Rotat East End	0	0	0	0	0
Measured Slip West End	3.2428		3.2424 ²⁴	3.2540	3.2540
Measured Rotat East End	0	0	0	0	0
Defl @ 42.0"	38 7/16	38 7/16	38 3/8	38 1/2	33 13/16
Defl @ 42.0" EAST END	37 3/4	37 5/16	37 3/4	38 1/8	39 1/2
THIS BM FAILED IN FLUXCORE - CONC REMAINING					
	CE ± 7600 lbf				

Miscellaneous Testing Data

Specimen:

A0.20-L

Date Tested:

Load	7.0 kip	7.5 kip			
Measured Defl ¼ Span	32 1/8	28 7/8			
Measured Defl ½ Span	32 7/8	30 7/16			
Measured Defl ¾ Span	35 1/4	34.0			
Defl @ West End	14 1/8	14 3/8			
	13 5/16	13 1/8			
Defl @ East End	13 15/16	13 7/8			
	14 1/16	14 1/16			
Bearing Contact West End	2 in A No Full	2 3/4 in			
Bearing Contact East End	Full	Full			
Measured Rotat West End	—	—			
Measured Rotat East End	—	—			
Measured Slip West End	3.2540	3.2540			
Measured Rotat East End	—	—			
	39.0	39 1/4			
	40 3/8	41 3/8			

Concrete Surface Strain

Specimen: B0.70-L
Date Tested: 10/30/13

Failure At 17.694 lb

Load		Prior Pile	0 lb	10,000 lb	15,000 lb	
1 to 3	8	0.21608	0.34388	0.34412	0.34163	
2 to 4	12	0.20722	0.18040	0.17941	0.17775	
3 to 5	16	0.25747	0.24011	0.23914	0.23913	
4 to 6	20	-	-	-	-	
5 to 7	24	0.116425	0.11650	0.14913	0.14325	
6 to 8	28	-	-	-	-	
7 to 9	32	0.25129	0.25783	0.25762	0.25635	1854 strain
8 to 9	36					
9 to 10	44	0.107489	0.08404	0.07047	0.08062	
10 to 11	52	0.34042	0.34753	0.34115	0.34002	
11 to 12	60	0.26087	0.20748	0.20475	0.19825	
12 to 13	68	0.33548	0.31135	0.34039	0.33642	
13 to 14	76	0.09821	0.08063	0.08799	0.07969	
14 to 15	84	0.24862	0.26049	0.25623	0.25533	
15 to 16	92	0.28730	0.27830	0.27855	0.27728	
16 to 17	100	0.14421	0.14964	0.15100	0.14759	
17 to 18	108	0.33802	0.34934	0.28675?	0.33654	
18 to 19	116	0.34936	0.33251	0.33013	0.32703	
19 to 20	124	0.12014	0.114378	0.13038	0.12920	
20 to 21	132	0.39113	0.37352	0.36955	0.36658	
21 to 22	140	0.16012	0.11320	0.10801	0.10469	
22 to 23	148	0.35980	0.36484	0.36313	0.36332	
23 to 24	156	0.11465	0.101031	0.10042	0.10042	
24 to 25	164	0.07284	0.06340	0.06196	0.05648?	
25 to 26	172	0.48695	0.48691	0.48062	0.48023	

Miscellaneous Testing Data

Specimen: B0.70-6
Date Tested: 10/30/13

[illegible]

Concrete Surface Strain

48" LOAD PT

Specimen: C-070-6
Date Tested: 10/25/13

Load	INITIAL	6000	8200 10000	# 12000	15000
21 1 to 2 8	0.00812	0.00534	0.00361	0.00228	0.00199
29 2 to 3 12	0.30751	0.30502	0.29695	0.3055 0.3035	0.29380
37 3 to 4 24	—	HOUSY COMB	—	—	—
45 4 to 5 32	—	—	—	—	—
53 5 to 6 40	0.41771	0.40749	0.40605 0.41091	0.40212	0.40038
61 6 to 7 48	0.25610	0.25016	0.24813	0.24159 0.25865	0.24146
69 7 to 8 56	—	REINFORCING STAKE INTERFERENCE	—	—	—
77 8 to 9 64	0.25572	—	—	—	—
85 9 to 10 72	0.25572	0.25772	0.25354	0.24106	0.23740
93 10 to 11 80	0.22704	0.22431	0.21990	0.21064	0.21092
101 11 to 12 88	0.36724	0.36479	0.36367	0.36183	0.35851
109 12 to 13 96	0.26932	0.26782	0.26562	0.26430	0.25805
13 to 14					
14 to 15					
15 to 16	POST #1 BOWS @ 16' 5/16" FROM END				
16 to 17	8" GAGE TAPE AFTER				
17 to 18					
18 to 19					
19 to 20					
20 to 21					
21 to 22					
22 to 23					
23 to 24					

1627 yds

Concrete Surface Strain

Specimen: C-0.70-6
Date Tested: 10/25/13

Load	17500	20000	22000		
1 to 2	0 less than 0				
2 to 3	0.30274	0.30242	0.30120		
3 to 4	—	—	—		
4 to 5	—	—	—		
5 to 6	0.39439	0.37619	0.29030		
6 to 7	0.23800	0.22812	0.19568		
7 to 8	—	—	—		
8 to 9	—	—	—		
9 to 10	0.24585	0.23345	0.21810		
10 to 11	0.20392	0.18713	0.15252		
11 to 12	0.35354	0.34597	0.33061		
12 to 13	0.25438	0.24455	0.22749		
13 to 14					
14 to 15					
15 to 16					
16 to 17					
17 to 18					
18 to 19					
19 to 20					
20 to 21					
21 to 22					
22 to 23					
23 to 24					

Flow Flex Bar
BULK

Miscellaneous Testing Data

Specimen: CO.70-6
Date Tested: 10/25/13

At End

At 16"

At 13 1/2"

Load	INITIAL	6000	8200	10000	12000
Measured Defl 1/4 Span	38 1/2	37 7/8	37 5/8	37 3/8	37 1/8
Measured Defl 1/2 Span	38 3/8	37 3/4	37 9/16	37 3/16	37 1/8
Measured Defl 3/4 Span	38 7/16	38 1/8	37 15/16	37 13/16	37 5/8
Defl @ West End Exterior	14 1/8	14 1/16	14 1/60	14 1/60	14 1/60
Defl @ West End Interior	38 1/2	38 1/4	38 3/16	38 1/8	38 1/16
Bearing Contact West End	FULL	FULL	FULL	FULL	FULL
Measured Slip West End	0.57 0.0574	0.0584	0.0585	0.0549	0.0575
Defl @ East End Exterior	13 13/16	13 13/16	13 13/16	13 13/16	13 13/16
Defl @ East End Interior	37 3/4	37 3/4	37 3/4	37 3/4	37 3/4
Bearing Contact East End	FULL	FULL	FULL	FULL	FULL
Measured Rotat East End					
LVDTs @ 1/4 span, 1/2 span, 3/4 span, 16" from East End					

C0.70-6

Date Tested: _____

2

Concrete Surface Strain

Specimen: DD-70-6
Date Tested: 11/2/13

Load	0lb	8.10K	12	16	20	21K
1 to 3	—					
2 to 4	—					
3 to 5	—	NOTE: 9 is 6" FROM END OF SPECIMEN				
4 to 6	—					
5 to 7	—					
6 to 8	—					
9 to 10	0.06073	0.08259	0.07213	0.07963	0.07929	0.08229
10 to 11	0.134401	0.34286	0.34211	0.33142	0.34103	0.34126
11 to 12	0.20169	0.20055	0.20484	0.19894	0.19956	0.20165
12 to 13	0.34045	0.34040	0.34248	0.34019	0.32052	0.29693
13 to 14	0.08210	0.08040	0.08058	0.06962	0.03306	0.00129
14 to 15	0.25843	0.25835	0.26150	0.25407	0.21611	0.18107
15 to 16	0.28293	0.27884	0.27900	0.27713	0.27753	0.28015
16 to 17	0.28214	0.15047				
17 to 18	0.15220	0.35677	0.14799	0.14402	0.13309	0.12592
18 to 19	0.33942	0.33651	0.33579	0.33676	0.33050	0.32544
19 to 20	0.33433	0.32853	0.33141	0.32935	0.32521	0.31848
20 to 21	0.12721	0.13 too loose				
21 to 22	0.11410	0.10000	0.11002	0.10934	0.11007	0.11140
22 to 23	0.37061	0.36711	0.36723	0.36852	0.36642	0.36724
23 to 24	0.09561	0.09794	0.11063	0.10353	0.09862	0.11041
24 to 25	0.106242	0.06057	0.05842	0.05960	0.06193	0.06102
25 to 26						

Miscellaneous Testing Data

Specimen: DD-170-6
Date Tested: 11/2/13

[illegible]

Concrete Surface Strain

Specimen: E070-L
Date Tested: 11/5/13

Load	0 lb	12,000 lb	21,000 lb	24,000 lb	
1 to 2	0.43194	0.43877	0.42407	0.42348	
2 to 3	0.19348	0.19205	0.18729	0.16982	
3 to 4	0.35391	0.35202	0.34569	0.34584	
4 to 5	0.17822	0.17682	0.16862	0.16959	
5 to 6	0.24063	0.21063	0.19804	0.19540	
6 to 7	0.24672?	0.25335	0.24993	0.24547	
7 to 8	0.29118	0.28809	0.27822	0.27716	
8 to 9	0.16197	0.14312	0.13108	0.12803	
9 to 10	0.35704	0.35713	0.35363	0.35359	
10 to 11	0.22755	0.22271	0.22783	0.22772	
11 to 12					
12 to 13					
13 to 14					
14 to 15					
15 to 16					
16 to 17					
17 to 18					
18 to 19					
19 to 20					
20 to 21					
21 to 22					
22 to 23					
23 to 24					

Miscellaneous Testing Data

Specimen: E0.70-b
 Date Tested: 11/5/13

@ 12"

@ 16"

Load	0 lb	12,000 lb	21,000 lb	24,000 lb	26,000 lb
Measured Defl 1/4 Span					
Measured Defl 1/2 Span	37 13/16	37 1 1/2	37 3/16	37	
Measured Defl 3/4 Span					
Defl @ West End Exterior	13 15/16	13 5/8	13 1/2	13 1/2	
Defl @ West End Interior	37 5/16	37 9/16	37 3/8	37 5/16	
Bearing Contact West End	full	full	full	full	
Measured Slip West End	no slip	no slip	1/2 mm	1/2 mm	
Defl @ East End Exterior	12	12	12	12	
Defl @ East End Interior	38 3/8	38 1/4	38 3/16	38 3/16	
Bearing Contact East End	full	full	full	full	
Measured Rotat East End					
	SHARP FAILURE @ ALMOST 26K				

Concrete Surface Strain

Specimen: A 8.70-8
Date Tested: 11-8-13

Load	0	7	11	15	18.5	
1 to 2 18 to 19	0.49450	0.49407	0.49255	0.49388	0.49395	0.49395
2 to 3 19 to 20	0.11680	0.11773	0.11668	0.11872	0.11305	
3 to 4 20 to 21	0.24521	0.26659	0.25279	0.26201	0.25639	
4 to 5 21 to 22	0.22763	0.22816	0.22680	0.22483	0.26705	0.22072
5 to 6 22 to 23	0.34574	0.36532	0.36480	0.36452	0.34221	
6 to 7 23 to 24	0.19522	0.19448	0.19393	0.19059	0.18718	
7 to 8 24 to 25	0.11319	0.10732	0.11462	0.10968	0.109459	
8 to 9						
9 to 10	21K	23K	No cracks			
10 to 11 18 to 19	0.49395	0.49435	11K			
11 to 12 19 to 20	0.10226	0.08818				
12 to 13 20 to 21	0.24334	0.22867				
13 to 14 21 to 22	0.20639	0.19502				
14 to 15 22 to 23	0.33874	0.30738				
15 to 16 23 to 24	0.15718	0.07978				
16 to 17 24 to 25	0.05457	> 0.00000				
17 to 18						
18 to 19						
19 to 20						
20 to 21						
21 to 22						
22 to 23						
23 to 24						

Concrete Surface Strain

Specimen: B0.70-8
Date Tested: 11/26/13

POINT #1 - 4 3/16 FROM END

6

Load	OK	11 kips	18 kips		21 kips
1 to 2	0.31726	0.31613	0.31502		0.31632
2 to 3	0.09168	0.09127	0.08 0.09130		0.09049
3 to 4	0.30603	0.30594	0.30749		0.30488
4 to 5	0.31530	0.30842	0.30842		0.30849
5 to 6	0.04329	0.04264	0.03149		0.04242
6 to 7					
7 to 8	0.33631	0.32799	0.32773		0.32748
8 to 9	0.28292	0.28241	0.28308		0.28003
9 to 10	0.20069	0.19709	0.19410		0.19764
10 to 11	0.26050	0.25980	0.25989		0.25905
11 to 12	0.110880	0.10633	0.10578		0.09377
12 to 13	0.47803	0.47419	0.47348		0.47283
13 to 14	0.15186	0.14640	0.15682		0.14589
14 to 15	0.28398	0.28247	0.27863		0.27883
15 to 16	0.20531	0.20160	0.190 0.20043		0.20023
16 to 17	0.31595	0.319 0.31622	0.31995		0.31590
17 to 18	0.36135	0.35797	0.37624 0.35560 0.35560		0.35552
18 to 19	0.11356	0.11245	0.11410		0.11821
19 to 20	0.28764	0.28669	0.28705		0.28724
20 to 21	0.29300	0.28963	0.28867		0.28800
21 to 22	0.21683	0.21528	0.21328		0.21202
22 to 23	0.27036	0.27080	0.26593		0.26734
23 to 24	0.19734	0.19389	0.19322		0.19388

0.25905

~~24 to 25~~

Concrete Surface Strain

Specimen: B070-8
Date Tested: _____

Load	23 Kips	25 Kips			
1 to 2	0.31970	0.31176			
2 to 3 2-4	0.09295	0.08885			
3 to 4	0.30600	0.30492			
4 to 5	0.30990	0.30850			
5 to 6	0.04288	0.03549			
6 to 7	0.32734				
7 to 8	0.30734	0.32552			
8 to 9	0.27893	0.27590			
9 to 10	0.20714	0.19591			
10 to 11	0.26462	0.25463			
11 to 12	0.10313	0.10163			
12 to 13	0.47313	0.47048			
13 to 14	0.14409	0.14994			
14 to 15	0.27775	0.27731			
15 to 16	0.20076	0.19784			
16 to 17	0.31578	0.31638			
17 to 18	0.35533	0.29722	0.35722		
18 to 19	0.11588	0.11421			
19 to 20	0.28661	0.28328			
20 to 21	0.28710	0.28639			
21 to 22	0.21181	0.21162			
22 to 23	0.26418	0.26580			
23 to 24	0.19583	0.19160			

Miscellaneous Testing Data

Specimen: B070-8
Date Tested: 11/26/13

[illegible]

BO: 10 - 8

Date Tested:

236

Concrete Surface Strain

Specimen:
Date Tested:

Specimen A0.70-10
1/27/14

10.3
1.82
4

Load	6K	9K	13K	17K	20K	20.7K
1 to 2	0.23176	0.22926	0.22929	0.23024	0.22807	0.23111
2 to 3	0.20560	0.20215	0.20301	0.20283	0.20393	0.20325
3 to 4	0.14621	0.15294	0.15315	0.15421	0.15485	0.15455
4 to 5	0.06049	0.25863	0.25849	0.25733	0.25857	0.05213
5 to 6	0.15543	0.15628	0.15631	0.15466	0.15808	less than 0
6 to 7						
7 to 8	0.30547	0.30170	0.29020	0.29071	0.28950	0.29117
8 to 9	0.17673	0.17297	0.17261	0.16962	0.16799	0.16689
9 to 10	0.43815	0.43465	0.43448	0.43388	0.43534	0.43362
10 to 11	0.47774	0.47656	0.47380	0.47467	0.47255	0.48454
11 to 12	—	—	—	—	—	
12 to 13	—	—	—	—	—	
13 to 14	—	—	—	—	—	
14 to 15	—	—	—	—	—	
15 to 16	0.08675	0.08487	0.08562	0.08461	0.08604	0.08561
16 to 17	0.40575	0.40478	0.40264	0.40011	0.39377	0.39288
17 to 18	0.21459	0.21338	0.21323	0.21241	0.21315	0.21429
18 to 19	0.32162	0.32291	0.32133	0.31420	0.30477	0.30824
19 to 20	0.34180	0.34110	0.33844	0.33158	0.32922	0.32559
20 to 21	0.17117	0.16664	0.16549	0.16260	0.15664	0.15398
21 to 22	0.23663	0.23856	0.23676	0.22869	0.22764	0.22589
22 to 23	0.27113	0.26728	0.26853	0.26427	0.25566	0.25551
23 to 24	0.02378	0.02047	0.02092	0.02208	0.02120	0.02120
24 to 25	0.37061	0.37282	0.37221	0.36522	0.35842	0.35564

Miscellaneous Testing Data

Specimen: Specimen A070-10
Date Tested: 1/27/14

Date Tested: 11/27/14
BMY West

[illegible]

Concrete Surface Strain

Specimen:

Date Tested:

~~610~~ B070-10

1/31/14

Beam 4 East

Load	0K	15K	18K	21K	22.5K	24K
1 to 3	0.16624	0.16532	0.16520	0.15135	0.16373	0.16212
2 to 4	0.43818	0.43275	0.43179	0.43201	0.43394	0.43125
3 to 5	0.41932	0.41910	0.40903	0.40464	0.40870	0.41014
4 to 6	0.14411	0.09039	0.10231	0.10087	0.0965	0.10254
5 to 7	0.09411	—	—	—	—	—
6 to 7	—	—	—	—	—	—
7 to 8	0.22469	0.22021	0.21563	0.20152	0.21261	0.18206
8 to 9	0.49979	0.50132	0.49731	0.49729	0.49711	0.48237
9 to 10	0.19486	0.19335	0.18672	0.18012	0.16931	0.12967
10 to 11	0.13389	0.13017	0.12553	0.11863	0.11541	0.08763
11 to 12	0.24986	0.24640	0.24338	0.23816	0.23355	0.22120
12 to 13	—	—	—	—	—	—
13 to 14	—	—	—	—	—	—
14 to 15	0.32201	0.32944	0.31343	0.30979	0.30458	0.29164
15 to 16	0.15089	0.15897	0.15634	0.15159	0.14762	0.14078
16 to 17	0.20249	0.27011	0.26982	0.27436	0.27389	0.27365
17 to 18	0.22681	0.23758	0.23049	0.22347	0.22659	0.22292
18 to 19	0.38332	0.37657	0.37837	0.37551	0.37190	0.36675
19 to 20	0.35002	0.37655	0.33720	0.34662	0.34372	0.34859
20 to 21	0.21783	0.22200	0.21602	0.21466	0.21213	0.20067
21 to 22	0.30601	0.30188	0.29888	0.29753	0.29706	0.29758
22 to 23	0.17549	0.17050	0.17295	0.17265	0.17213	0.17412
23 to 24	0.15888	0.16079	0.16334	0.16355	0.15312	0.16406
24 to 25	0.23722	0.23512	0.23568	0.23592	0.23835	0.23237
25 to 26	0.22086	0.23641	0.23695	0.23541	0.23785	0.23830

Concrete Surface Strain

Specimen:

80.20-10

Date Tested:

Load	24.7K	25.2K			
1 to 2	0.15125	0.16466			
2 to 4	0.43367	0.43462			
3 to 5	0.40615	0.40954			
4 to 6	0.09001	0.10063			
5 to 6					
6 to 7					
7 to 8	0.16574	0.13942			
8 to 9	0.47103	0.46804			
9 to 10	0.09012	0.05230			
10 to 11	0.06874	0.04847			
11 to 12	0.24337	0.19640			
12 to 13					
13 to 14					
14 to 15	0.28318				
15 to 16	0.13514				
16 to 17	0.27104				
17 to 18	0.21588				
18 to 19	0.35631				
19 to 20	0.34113				
20 to 21	0.20938				
21 to 22	0.29527				
22 to 23	0.17380				
23 to 24	0.15684				

24-25
25-26

0.23120
0.22756

Miscellaneous Testing Data

Specimen: . B0.70-10
Date Tested: 1/31/14

[illegible]

Concrete Surface Strain

79" load Pt

Specimen: A0.62-6
Date Tested: 09/26/2013

No crack. Cracked

Load	0	3 kips	8 kips	At Failure	Prior to Release
1 to 3	—	—	—	—	0.34767
2 to 4	0.09838	0.10438	0.09442	0.09781	0.08859 0.08859
3 to 5	0.09462	0.09582	0.09569	0.11000	0.01682
4 to 6	0.18474	0.17726	0.17223	0.17973	0.16892
5 to 7	0.17841	0.25501	0.26829	0.20567	0.25491
7 to 8	0.36423	0.3381	0.32889	0.32762	0.32700
8 to 9	0.25259	0.28874	0.26163	0.26567	0.26947
9 to 10	0.23823	0.25370	0.24527	0.23160	0.22058
10 to 11	0.10613	0.10753	0.09109	0.09212	0.09656
11 to 12	0.21538	0.22210	0.20910	0.21089	0.21604
12 to 13	—	—	—	—	—
13 to 14	—	—	—	—	—
14 to 15	0.23715	0.23764	0.21867	0.20683	0.21849
15 to 16	0.18555	0.20627	0.18452	0.18865	0.18772
16 to 17	0.32157	0.27920	0.23604	0.30175	0.31003
17 to 18	0.29496	0.28829	0.26624	0.29290	0.28845
18 to 19	0.12600	0.10895	0.09633	0.09663	0.10213
19 to 20	0.41568	0.41661	0.40076	0.41290	0.42394
20 to 21	0.21914	0.20106	0.18682	0.20184	0.20466
21 to 22	0.33761	0.33873	0.31904	0.33432	0.33116
22 to 23	0.17325	0.17489	0.14270	0.16422	0.17714
23 to 24	0.34713	0.34024	0.32446	0.33221	0.33722
24 to 25	0.1088	0.15000	0.12821	0.15430	0.15861

6 kips start crack.

21052

Concrete Surface Strain

Specimen: C0.62-6 (Pam. 3 Module)
Date Tested: 10/8/13

10-5 load point

After Cut

6107
↓

Load	INITIAL	INITIAL ^{2ND}	2000lb	2000lb	4000lb
1 to 2	0.19041	0.18570	0.18783		0.18857
2 to 3	0.29264	0.29203	0.29325		0.29208
3 to 4	0.29118	0.29154	0.29456*	0.29888	0.29763
4 to 5	0.10983	0.10630	0.10658		0.11175
5 to 6	0.40550	0.40540	0.40669		0.40641
7 to 8	0.14439	0.14476	0.14872		0.14672
8 to 9	0.36092	0.35803	—	—	
9 to 10	0.19175	0.19160	0.19512	0.19473	0.19453
10 to 11	0.30876	0.31184	0.31433	0.31410	0.30982
11 to 12	0.26193	0.26072	0.25205	0.25578	0.25215
12 to 13	0.26401	0.26217	0.27209	0.26735	0.26532
13 to 14	0.22528	0.22532	0.22250*	0.22800	0.22162
14 to 15	0.27291	0.2708	0.27213	0.27180	0.27276
15 to 16	0.27836	0.27625	0.27923	0.27620	0.28091
16 to 17	0.20975	0.20812	0.20294*	0.20598	0.19913
17 to 18	0.16310	0.16348	0.16395	0.16234	0.16182
18 to 19	0.45842	0.46079	0.45279*	0.45261	0.45735
19 to 20	0.15091	0.15020	0.14883	0.14930	0.14822
20 to 21	0.17772	0.17753	0.18734*	0.19180	0.19385
21 to 22					
22 to 23					
23 to 24					
24 to 25			1.359		

650-410-6123

Concrete Surface Strain

Specimen: C0,62-6
Date Tested: 10/13/13

Load	8000lb	10,000lb			
1 to 3	0.19873	0.19673			
2 to 4	0.29062	0.28950			
3 to 5	0.28697	0.29895			
4 to 6	0.11180	0.10495			
5 to 7	0.40682	0.40087			
7 to 8	0.14765	0.14632			
8 to 9					
9 to 10	0.18532	0.18726			
10 to 11	0.31405	0.30735			
11 to 12	0.25368	0.25700			
12 to 13	0.26723	0.27045			
13 to 14	0.20648	0.20662			
14 to 15	0.26988	0.26800			
15 to 16	0.27135	0.26245			
16 to 17	0.19635	0.19850			
17 to 18	0.15145	0.14600			
18 to 19	0.45952	0.44735			
19 to 20	0.14032	0.13115			
20 to 21	0.18684	0.18266			
21 to 22					
22 to 23					
23 to 24					
24 to 25					

Miscellaneous Testing Data

Specimen:

C062-6

Date Tested:

After COT

[illegible]

Concrete Surface Strain

Specimen: Specimen A 0.628
Date Tested: 12/16/13

Load	0lb	6 kips	10 kips	12 kips	14.5 kips
1 to 2	0.26943	0.26945	0.27150	0.27177	0.27200
2 to 3	0.29238	0.29468	0.29252	0.28978	0.29153
3 to 4	0.18312	0.18280	0.18285	0.18244	0.18323
4 to 5	0.14201	0.14069	0.14107	0.14012	0.14000
5 to 6	0.32873	0.32561	0.31712	0.32548	0.31000
6 to 7	0.25979	0.25859	0.24853	0.26081	0.24659
7 to 8	0.29439	0.29488	0.29490	0.29357	0.29446
8 to 9	0.24158	0.24295	0.24209	0.23750	0.23795
9 to 10	0.05000	0.04720	0.04907	0.04488	0.04482
10 to 11	0.35715	0.35256	0.35456	0.35096	0.34657
11 to 12					
12 to 13					
13 to 14					
14 to 15					
15 to 16					
16 to 17					
17 to 18					
18 to 19					
19 to 20					
20 to 21					
21 to 22					
22 to 23					
23 to 24					

Concrete Surface Strain

Specimen: ① A062-8
 Date Tested: _____

Load	15.8 kips	15.9 kips	Fail	Final	
1 to 2	0.27027	0.26762	0.27250		
2 to 3		0.28971	0.29295		
3 to 4		0.18310	0.18383		
4 to 5		0.13988	0.13962		
5 to 6		0.25286	0.01855		
6 to 7		0.24057	0.24656		
7 to 8		0.29411	0.24506		
8 to 9		0.23080			
9 to 10		0.04450			
10 to 11		0.34220			
11 to 12					
12 to 13					
13 to 14					
14 to 15					
15 to 16					
16 to 17					
17 to 18					
18 to 19					
19 to 20					
20 to 21					
21 to 22					
22 to 23					
23 to 24					

Miscellaneous Testing Data

Specimen: Specimen A0.628
 Date Tested: 12/16/13

Load	0lb	6 kips	10 kips	12 kips	14.5 kips
Measured Defl ¼ Span	37 1/16	36 3 5/16	36 1/8	35 3/4	34 7/8
Measured Defl ½ Span	37 1/2	36 3/4	36 3/8	36	35
Measured Defl ¾ Span	37 3/4	37 11/16	37 3/8	37 1/8	36 7/16
Defl @ West End Exterior	14 1/4"	14 3/16	14 1/8	14 3/16	14 1/4
Defl @ West End Interior	37 7/8	37 11/16	37 7/16	37 3/8	37 1/8
Bearing Contact West End	Full	Full	Full	Full	Full
Measured Slip West End					
Defl @ East End Exterior	13 15/16	14	14	13 13/16	14
Defl @ East End Interior	37 5/16	37 7/8	37 3/4	37 3/4	37 5/8
Bearing Contact East End	Full	Full	Full	Full	Full
Measured Rotat East End					
LOAD PT	37 1/4	37 1/8	36 7/16	36 1/2	36

Concrete Surface Strain

Specimen: Specimen B062-8
Date Tested: 1/22/14

Load	0K	10K	12.8		
1 to 3	0.21674	0.20961	0.21491		
2 to 4	0.07203	0.07769	0.07189		
3 to 5	0.19664	0.19563	0.19114		
4 to 6	—	—	—	—	—
5 to 7	0.15411	0.158 0.15758	0.15681		
6 to 8	0.14590	—	—	—	—
7 to 8	0.14590	0.25005			
8 to 9	0.27128	0.27066			
9 to 10	0.119379	0.19058			
10 to 11	0.166969	0.06677			
11 to 12	0.148593	0.48627			
12 to 13	0.13910	0.13885			
13 to 14	0.28458	0.28865			
14 to 15	0.132969	0.32822			
15 to 16	0.105663	0.05373			
16 to 17	—	—	—	—	—
17 to 18	0.25578	0.25054			
18 to 19	0.10130	0.09751			
19 to 20	0.46055	0.45833			
20 to 21	0.12891	0.13940			
21 to 22	0.25164	0.24914			
22 to 23	0.20505	0.20492			
23 to 24	—	—	—	—	—

24 to 25

— — — — —

Miscellaneous Testing Data

Specimen: Specimen BD, 62-8

Date Tested: 1/22/24

12^u @

18th @

Concrete Surface Strain

Specimen: C0.62-8
 Date Tested: 12/3/14

Load	0	11 kips	17 kips	23 k	
1 to 2	0.21344	0.21278	0.21316	0.21414	
2 to 3	0.33473	0.32409	0.32275	0.31534	
3 to 4	0.15807	0.15634	0.15308	0.14522	
4 to 5	0.20248	0.21242	0.19785	0.15789	
5 to 6	0.26951	0.26655	0.26512	0.25574	
6 to 7	0.31470	0.31812	0.31789	0.31189	
7 to 8	0.19483	0.19211	0.19256	0.19091	
8 to 9	0.32623	0.32358	0.32282	0.31337	
9 to 10	0.28352	0.285 0.28571	0.28187	0.27640	
10 to 11					
11 to 12					
12 to 13					
13 to 14					
14 to 15					
15 to 16					
16 to 17					
17 to 18					
18 to 19					
19 to 20					
20 to 21					
21 to 22					
22 to 23					
23 to 24					

Miscellaneous Testing Data

Specimen: Specimen 6A62-6
Date Tested: _____

[illegible]

Miscellaneous Testing Data

Specimen: Specimen A062-10
Date Tested: 1/23/14

[illegible]

Concrete Surface Strain

Specimen: Specimen BO.62-10
Date Tested: 1/24/14

Load	OK	6K	9K	12K	
1 to 2	—	—	—	—	—
2 to 3	0.39076	0.39357	0.39024	0.39044	
3 to 4	0.41664	0.41242	0.41140	0.41099	
4 to 5	0.23733	0.23268	0.23226	0.23201	
5 to 6	0.14805	0.15022	0.15294	0.15301	
6 to 7					
7 to 8	—	—	—	—	—
8 to 9	0.05505	0.05567	0.04521	0.05539	
9 to 10	0.145277	0.45293	0.45264 0.45260	0.4525	
10 to 11	0.12759	0.02415	0.022407	0.02613	
11 to 12	0.30210	0.29901	0.29880	0.29513	
12 to 13	0.29248	0.30257	0.30504	0.30372	
13 to 14	0.10469	0.10508	0.11225	0.10362	
14 to 15	0.31765	0.31792	0.32350	0.31638	
15 to 16	0.17458	0.17300	0.17368	0.17183	
16 to 17	0.42843	0.43050	0.42987	0.43896	
17 to 18	0.22503	0.22127	0.22520	0.22202	
18 to 19	0.14230	0.14000	0.14217	0.14871	
19 to 20	0.23773	0.23508	0.24077	0.24442	
20 to 21	0.37975	0.37952	0.38200	0.38472	
21 to 22	0.16343	0.16632	0.16582	0.17264	
22 to 23	0.25422	0.25576	0.25581	0.25953	
23 to 24	0.36213	0.36287	0.36348	0.36799	

24 to 25 0.15704 0.15852 ~~0.15386~~ 0.15180

Miscellaneous Testing Data

Specimen:

Date Tested:

Specimen BO.62-10

12414

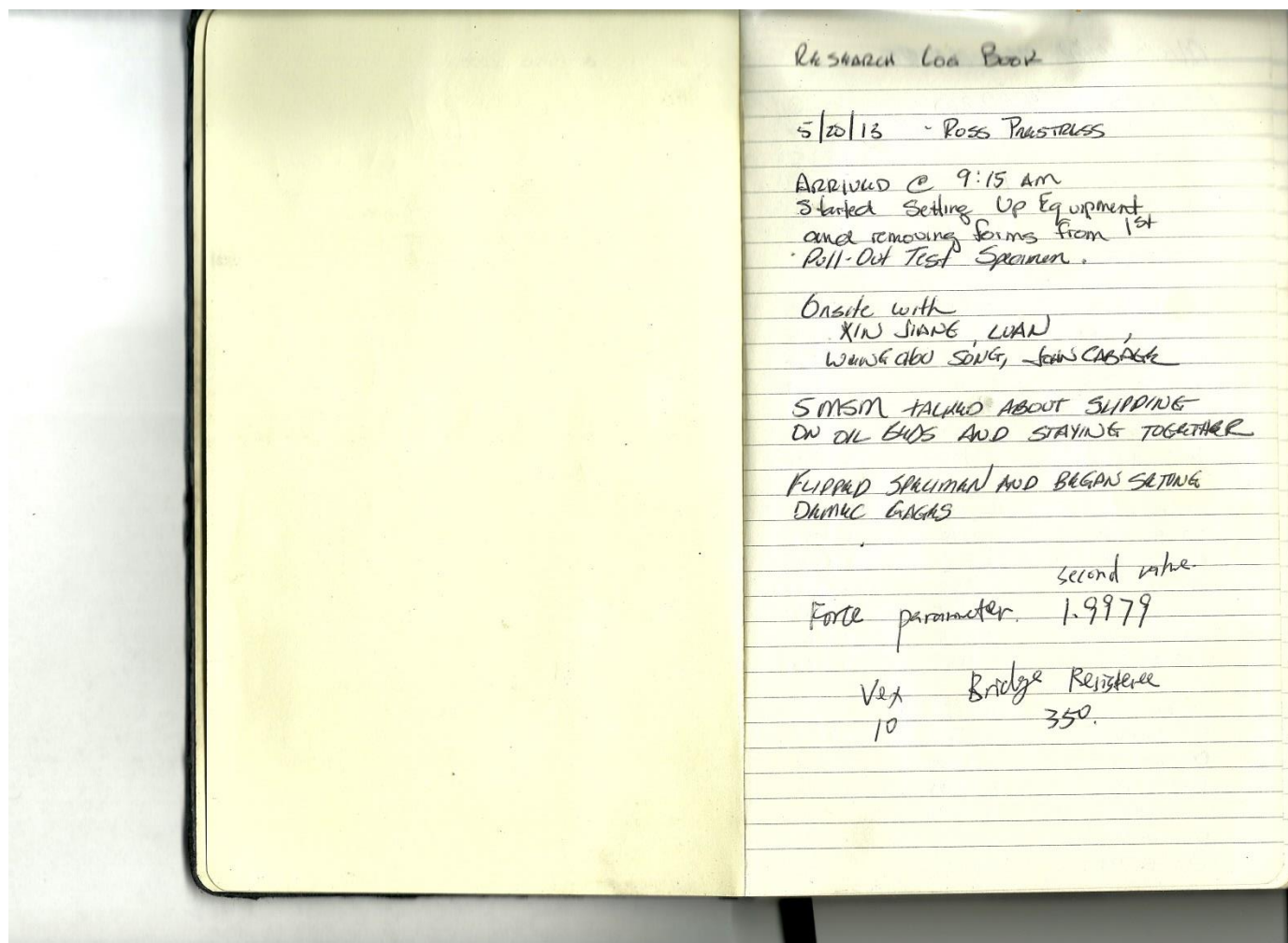
[illegible]

16th

1866

~~Hostip~~ slip

Appendix D – Field Notebook



MICROMETER MEASUREMENTS (mm)

A1 to A3	4.0020	3.9295
A2 to A4	3.9365	3.8970
A3 to A5	4.0400	3.9835
A4 to A6	4.0310	3.9835
A5 to A7	3.9460	3.9295
A6 to A8	3.9660	3.9300
A7 to A9	3.9835	3.9630

B1 to B3	4.0155	3.9675
B2 to B4	3.9365	3.9000
B3 to B5	3.9410	3.8995
B4 to B6	4.0010	3.9855
B5 to B7	3.9950	3.9635
B6 to B8	3.9620	3.9250
B7 to B9	4.0115	3.9890

C1 to C3	4.0165	3.9895
C2 to C4	4.0165	3.9890
C3 to C5	3.9350	3.9020
C4 to C6	3.9630	3.9590
C5 to C7	3.9965	3.9950
C6 to C8	3.9615	3.9600
C7 to C9	3.9650	3.9600

D1 to D3	4.0095	
D2 to D4	3.9945	
D3 to D5	3.9280	
D4 to D6	3.9825	
D5 to D7	3.9825	
D6 to D8	3.9305	
D7 to D9	4.0130	

E1 to E3	4.0130
E2 to E4	3.9860
E3 to E5	3.8940
E4 to E6	3.9795
E5 to E7	4.0410
E6 to E8	3.9315
E7 to E9	3.9630

F1 to F3	4.0250
F2 to F4	3.9740
F3 to F5	3.9600
F4 to F6	4.0490
F5 to F7	4.0025
F6 to F8	3.8990
F7 to F9	4.0135

Free end slip $2.4260 / 2.6415$

Tacket end slip 0.9375 (2.5 Kg)

Form	2.6	4.2	7.2	12.0	22.0
A1-A2	3.9370	3.9480	3.9445	3.9370	3.9370 3.9395
A2-A4	3.9405	3.9495	3.9385	3.8990	3.8915
A4-A5	3.9770	3.9945	3.9810	3.9780	3.9925
A4-A6	3.9770	3.9810	3.9810	3.9280	3.9925
A5-A7	3.9300	3.9240	3.9255	3.9220	3.9275
A6-A8	3.9300	3.9240	3.9260	3.9265	3.9275
A7-A9	3.9650	3.9750	3.9635	3.9655	3.9585

B1-B3
B2-B4
B3-B5
B4-B6
B5-B7
B6-B8
B7-B9

Free end slip 2.6810 2.6630

Jack slip 1.0020 1.0420 1.2200 1.3070 1.4535

LVDI

FINISHED TESTING WITH THREADS
STRIPPED OFF $\frac{1}{8}$ " ϕ ROD
AND NOT WITH A LARGER
MAD DRILL FROM KNU THOMAS

	58/55	63/61	50
3.0.0	43.0	51.0	57.0
3.9410	3.9385	3.9365	3.9360
3.8940	3.8965	3.8925	3.8940
3.9805	3.9845	3.9870	3.9920
3.9805	3.9845	3.9250	3.9815
3.9160	3.9225	3.9175	3.9220
3.9315	3.9275	3.9280	3.9340
3.9595	3.9630	3.9575	3.9705
			3.9400
			3.9455
			3.9040
			3.9905
			3.9930
			3.9840
			3.9750
			3.9290
			3.9250
			3.9465
			3.9385
			3.9670
			3.9720

2.6800 2.6600 2.6270 2.6035 2.5900 2.5815

1.4845 1.5975 1.7100 1.8115 1.9540 2.0175

0.015 0.017

5/11 TOOK PICTURE OF
BACK LOGS

5/16 WENT WITH VISITING SCIENTISTS
TO ROSS PRESSURE. BOULD
FIRST SPECIMEN SAMPLE AND
POURED CONCRETE AND 4
CYLINDER SPECIMENS.

4" ϕ - 8" TALL SPECIMENS
CAST.

~~WENT~~ INDIVIDUALS AT SITE
WU WEI, DR WEI, YUAN JING
DR LI, DR HO, XIAO JING BUR

5/17 SPENT TIME WITH K
THOMAS FINALIZING MANDREL
- 1 1/2 HR
STUDIED MODELING

5/19 READ PAPER FROM BURLY
GAINS

5/20 TESTED SPECIMEN ON
PREVIOUS PAGES

5/21 MET DR MA
~~REVIEW~~ LIT REVIEW OF 0.7-inch
model

5/22 SPENT MOST OF THE DAY
WITH DEREK RAY OF NTS TRYING
TO DETERMINE REQUIREMENTS
OF HYDRAULIC CYLINDERS FOR
TESTING. - TDOT GAR

P/D MANDREL @ ROSS PRESSURE
FOR MEN. CLEANED THREADS
WHICH WERE DAMAGED DURING
TESTING - 1 HR

5/23 REVIEWED PAPERS THAT DR MA
GAVE ME ON MODIFIED COMPRESSION
THEORY FROM BOOK WRITTEN BY
COLINS & MITCHELL - TDOT GAR

5/23 - P/D MANDREL FROM
K THOMAS. COMPLETED LOG BOOK

5/23

ERROR 38705 OCCURRED AT
DAQ My Create Channel (AI-Positron
LUDT) v1.2

Possible reason the specified
device is not present or is not
active in the system. The device
may have been unplugged or may
not have been installed correctly
Device SCIMat 6

Task Name ON panel task (4)

Force	13.4/12.9	20.6	26.3	36.3
E1-E3	4.0775	4.0170	4.0165	4.0170
E2-E4	4.0425	3.9815	3.9955	3.9915
E3-E5	3.9515	3.8945	3.8930	3.8920
E4-E6	4.0515	3.9860	3.8940	3.9850
E5-E7	4.1060	4.0535	4.0535	4.0508
E6-E8	3.9885	3.9295	3.9335	3.9195
E7-E9	4.0240	3.9640	3.9710	3.9660

Freehand SLP
CYL DSP

~~Break~~ Pop Occurred At about
604 K Fall back to 45.5.
Ran Test to failure
Removed Apparatus Localized Spalling
at entrance of strand Strand broke
at failure

Break
Occurred

40.6	49.5	56.5
4.0130	4.0185	4.0125
3.9855	3.9925	3.9835
3.8800	3.8870	3.8930
3.9915	3.9840	3.9858
4.0480	4.0480	4.0435
3.9275	3.9220	3.9305
3.9590	3.9610	3.9555

Force	20.5/19.9	33.1/31.9	46.2
E1-E3	4.0225	4.0220	4.0220
E4-E6	3.9465	3.9470	3.9505
E7-E9	4.0535	4.0565	4.0540
E10-E12	4.0035	4.0055	4.0015
E13-E15	3.9020	3.9075	3.9025
E16-E18	4.0100	4.0230	4.0155

DCYL 1.0965 1.1735 1.3740

Tensioned Single Specimen #1 to failure
Strand Break Failure C Similar
Strand Strength Localized
Spalling but no observable cracking
or bond slip

Force	24.2/23.1	40.4/40.3	54.7/53.9	69.5/68.5
D1-D3	3.9935	3.9965	4.0085	3.9920
D2-D4	3.9915	3.9710	3.9850	3.9765
D3-D5	3.9230	3.9235	3.9215	3.9220
D4-D6	3.9840	3.9905	3.9915	3.9955
D5-D7	3.9790	3.9750	3.9815	3.9830
D6-D8	3.9375	3.9335	3.9335	3.9315
D7-D9	4.0130	4.0100	4.0285	4.0230
CYLINDR	1.4595	1.5880	1.6915	1.9215

Tensioned Specimen to 100k Hard Pop
Reflexion to breakage concrete cracked
Immediately, Bond broke first and
we had a residual to $\pm 45K$

A crack formed on both sides of the specimen
we repressed the specimen and
the double strand slipped
and the concrete separated
and tensile force dropped to
zero, A rotational displacement
was noted

In addition to the concrete breaking
1/2 the strands broke near ultimate
strength, this may account for the
first popping noise at ultimate strength
Some of the strands remain bonded
during the test after separating the
concrete blocks. I need to
study what induced the tension in
the concrete. I could have been

51.4/50.3	93.3/92.1	the compression
4.0015	3.9965	of the concrete
3.9715	3.9685	by the cylinder
3.9250	3.9325	load
3.9850	3.9840	34 photos of
3.9850	3.9845	broken strands etc.
3.9325	3.9355	
4.0115	4.0210	
1.9405	2.1045	

CALIBRATION OF 150K LOAD CELL

WIRE CONNECTIONS

G - Pin Connector Side

A - Excitation +	OW DW	2 6
B - Remote Sense +	OW B	1 4
C - Excitation -	OW BW	1 7
D - Remote Sense -	OW G	5 5
E - Analog Output +	OW O	2 2
F - Analog Output -	OW GW	3 3

Tan - Pin RJ45 Connection into
NI 9237 Control Module

# Pin	Wire Color	for green connectors	Type
1)	Purple	SC	Shunt Calibration
2)	Blau	AL+	Analog Output +
3)	Orange	AL-	Analog Output -
4)	Black	RS+	Remote Sensor +
5)	Red	RS-	Remote Sensor -
6)	Green	EX+	Excitation +
7)	Yellow	EX-	Excitation -
8)	Brown	T+	TEOS System +
9)	Gray	T-	TEOS System -
10)	White	SC	Shunt Calibration

10V Excitation

Use 3.04 mV/V for Calibration

CALIBRATION OF 200K LOAD CELL

WIRE CONNECTION

4 WIRE SET

GREEN	- Excitation +
BLACK	- Excitation -
WHITE	- Analog Output +
RED	- Analog Output -

Connects to

Green	→	Position 6 G
Black	→	Position 7 GW
White	→	Position 2 BL
Orange	→	Position 1 (G) BLW

10V Excitation

Use 2.00 mV/V for Calibration

Load Cell CAPI

200K CAPACITY

SNR # 33484

BH ELECTRONICS

10 350 Ω BRIDGE

7/15/13

K2 45298

BOX TRUCK LICENSE
H2 4529B
PHEUP 263 3N1

YARD WORK

12405 Nuclear Industry

APR - Seabrook Nuclear Plant
Path to License

Structural Impact (Thick wall)
no shear reinforcement

3 Point Shear

Boundary conditions

- 1) Build a test cube for
mechanical cracking
- 2) Sufficient amount of water
Apply prestressing confinement
Proposed mid August

How Strong 23/32
TAP 5/8 coarse machine

~~3/4~~
200K | 25/32

Beam 1 1/4"

Flange of Yellow BM 1 1/8

150K - 3" of metal top 2 1/4 bot
200K - 2 3/4" of metal top
and 2 1/4" of metal bot

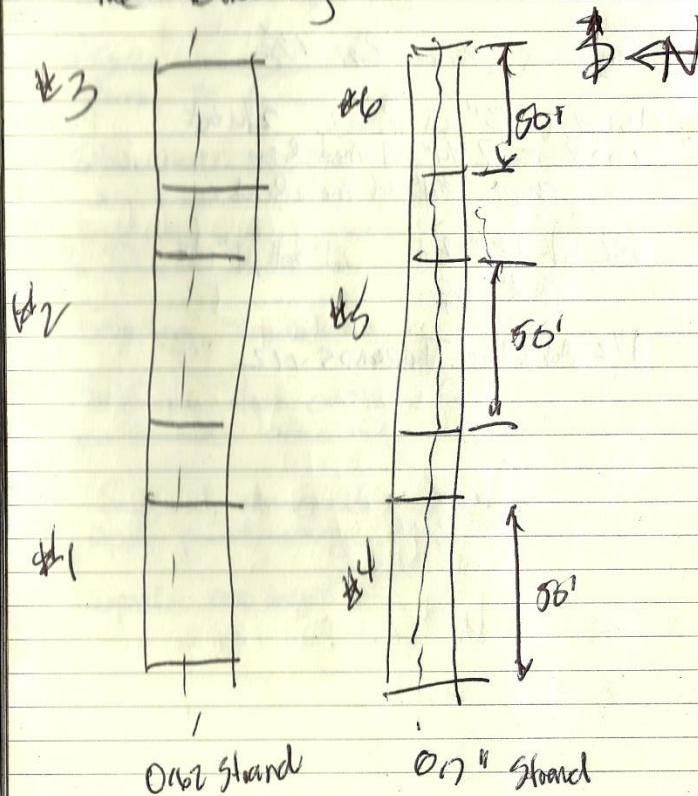
55K 4 1/4 2 1/4" bot 1" bot, 1" bot
4 1/4 2" top

1 1/2 Adaptor Tefreeds x 12

August 29, 2013

Set Strain Gauges on development Length Specimens at Ross Prestressed

The bed was laid out in the following fashion



Total Bed length = 222 ft

We pretensioned the strand to 5000 lb and then installed the strain gauges. The position was corrected for the additional extension of the strand

Specimen #	Strain Gauge type	Gf
1 W	C2A-03-250LW-350/c-331	211001
1 E		0.58
2 W		
2 E		
3 W	C2A-06-062LW-350	2.125
3 E	C2A-06-125LW-350	2.125
4 W	C2A-03-250LW-350/c-331	
4 E		
5 W		
5 E		
6 W		
6 E		

We installed per strain gauge locations and exposed in place. We taped the wires to the side of the form.

* A C2A-06-125LW-350 was used on the innermost strand gauge position

Specimen #2W 1st W

Replacement Gauge GA-06

- 062LW-350

Specimen #3W 1st W

GA-06-125LW-350

Specimen #3W 2nd W

GA-06-062LW-350

Specimen 1W 2nd W

C2A-06-125LW-350

Specimen 6E 5th W

C2A-06-125LW-350

Specimen 1E 5th W

C2A-06-125LW-350

9/3/13

I ARRIVED @ ROSS PRESTRAYS @ 7:45
LET THE CREW DRED THE BAYS.

I RECHECKED EACH STRAIN GAUGE
TO 350 SZ AND INSTALLED
STRAWS ON THE REMAINING
GAUGES. I REPAIRED 3 GAUGES
NOTED ON PREVIOUS PAGE.

WE SET 2 FORM BRAMS
AT SPECIMEN 3E SW AND 6E SW
THE MIX DESIGN IS TO BE
2500 PSI.

WE CHECK THE SIDE FORMS FOR
PLUMB, ~~AND~~ ~~ADJUST~~

Specimen Dist From End To
Stream Gauge

1W	1st	175 1/8
	2nd	120 1/2
	3rd	139 3/4
	4th	154 3/4
	5th	170 1/4
1E	1	18 1/2
	2	126
	3	140 3/8
	4	158 7/8
	5	172 5/8
2W	1	17 1/2
	2	123 7/8
	3	138 3/4
	4	153 3/4
	5	169 1/4
2E	1	17 3/4
	2	125 3/4
	3	140 3/8
	4	156 1/8
	5	171 1/4
3W	1	17 1/2
	2	125 3/8
	3	141
	4	155 3/4
	5	171 3/8
3E	1	18 1/2
	2	126 3/4
	3	141 1/4
	4	156 1/2
	5	172 1/4

4W	1	213 1/4
	2	124 3/8
	3	139 1/8
	4	154 7/8
	5	172 1/2
4E	1	21
	2	128 3/4
	3	144 3/4
	4	158 3/8
	5	175 1/4
5W	1	17 1/2
	2	125 3/8
	3	140 5/8
	4	155 3/8
	5	170 3/4
5E	1	19 1/8
	2	126 7/8
	3	142
	4	156 3/4
	5	172 1/8
6W	1	17
	2	124 1/2
	3	139 1/2
	4	154 3/8
	5	169 1/2
6E	1	18 1/2
	2	125 5/8
	3	141 3/8
	4	156 3/4
	5	171 5/8

High Capacity Strands Make
Thinner Bridges Longer

9/4/2013 Arrived @ 8:00 Sat
Forms STRIPPED

Went to UT to work

ARRIVED BACK at 3:30

Verified forms. Began Pours

@ 4:00. Finished Pouring 5:00

Concrete Specimens with

9/5 worked on BM 3 & 6

Demon Work, Exposed GUSON

Inserts, Check Formwork

Stripped forms for BM 2 & 5

Transferred forms to BM 1 & 4

Poured concrete BM 1 & 4

Cured with wet Burlap

Left site 6:00

9/6 Drying Insert Nodes BMS

1, 2, 4, 5 after forms Dewatered

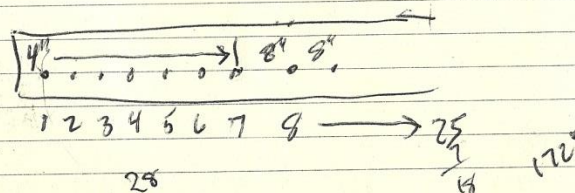
Exposed all inserts by 1:30

Took lunch and began

King GUSON Measurements

BM 3 EAST

1-3	0.155228
2-4	0.119965
3-5	0.122317
4-6	0.15513
5-7	0.11048
7-8	0.46880
8-9	0.26831
9-10	0.39859
10-11	0.40554
12-13	0.49579
13-14	0.19944
14-15	0.17610
15-16	0.42789
16-17	0.24978
17-18	0.12436
18-19	0.32200
20-21	0.49571
21-22	0.106271
22-23	0.39574
23-24	0.11632
24-25	0.29673
11-12	0.00268
19-20	
23-24	0.42681



BM 6 EAST

1-3 0.34608
 2-4 0.20722
 3-5 0.25747
 4-6 NO READING - TOO FAR APART
 5-7 0.16425
 7-8 0.25170
 8-9 0.07489
 9-10 0.34042
 10-11 0.26087
 11-12 0.33548
 12-13 0.09021
 13-14 0.24862
 14-15 0.28730
 15-16 0.14421
 16-17 0.33802
 17-18 0.34936
 18-19 0.12044
 19-20 0.39113
 20-21 0.16012
 21-22 0.35980
 22-23 0.11465
 23-24 0.07284
 24-25 0.48605
 25

BM 3 WEST

1-3 0.34767
 2-4 0.08859
 3-5 0.018682
 4-6 0.16802
 5-7 0.25491
 7-8 0.32700
 8-9 0.26947
 9-10 0.22048
 10-11 0.9565 0.09656
 11-12 0.1604
 12-13 NO READING - TOO FAR APART
 13-14 NO READING TOO CLOSE TOGETHER
 14-15 0.21949
 15-16 0.18772
 16-17 0.31003
 17-18 0.25445
 18-19 0.110213
 19-20 0.41394
 20-21 0.20416
 21-22 0.33116
 22-23 0.17714
 23-24 0.34722
 24-25 0.18861
 25 0.19115

BM 6 WEST

1-3	0.13538
2-4	0.21085
3-5	0.42450
4-6	0.14764
5-7	0.116133
6-8	0.06744
7-9	NO READING TOO CLOSE
8-10	0.27894
9-11	0.24905
10-12	0.12943
11-13	0.07458
12-14	0.39485
13-15	0.129339
14-16	0.41542
15-17	0.57779
16-18	0.45942
17-19	0.36600
18-20	0.27768
19-21	0.15462
20-22	0.27846
21-23	0.41449
22-24	0.13605
23-25	0.18142

BM 376 Measurement,
taken 9/6/13 PM

BM 2 EAST

1-3	0.21000
2-4	0.10312
3-5	0.19482
4-6	0.49057
5-7	0.15082
6-8	0.26374
7-9	0.28232
8-10	0.18995
9-11	0.05995
10-12	0.49106
11-13	0.12373
12-14	0.29694
13-15	0.33507
14-16	0.04624
15-17	NO READING TOO CLOSE
16-18	0.123142
17-19	0.10312
18-20	0.28293
19-21	0.13205
20-22	0.24182
21-23	0.19081
22-24	NO READING TOO FAR APART
23-25	NO READING TOO CLOSE

Bm 5 EAST

1-3 0.31166
 2-4 0.09332
 3-5 0.30778
 4-6 0.30587
 5-7 0.20232
 7-8 0.24195
 8-9 0.25985
 9-10 0.22289
 10-11 0.27519
 11-12 0.11048
 12-13 0.44487
 13-14 0.16860
 14-15 0.27913
 15-16 0.20443 0.21062
 16-17 0.22403
 17-18 0.35712
 18-19 0.09663
 19-20 0.29618
 20-21 0.30349
 21-22 0.22735
 22-23 0.25802
 23-24 0.20917
 24-25 25 FORWARD GLAND IN
 0 BACK WARD - NO READING

Bm 2 WEST

1-3 0.08237
 2-4 0.40581
 3-5 0.43032
 4-6 0.27643
 5-7 0.20534
 7-8 0.26138
 8-9 0.13882
 9-10 0.40756
 10-11 0.08870
 11-12 0.33981
 12-13 } 13 Not Approxed IN, I Missed It
 13-14 }
 14-15 0.17039
 15-16 0.40742
 16-17 0.24375
 17-18 0.09370
 18-19 0.26972
 19-20 0.108157
 20-21 0.22567
 21-22 0.29772
 22-23 0.19450
 23-24 0.15723
 24-25 0.34522

BM 5 WEST

1-3 0.26697
 2-4 0.24773
 3-5 0.11420
 4-6 0.29068
 5-7 0.50482
 7-8 TOO FAR APART - NO READING
 8-9 TOO CLOSE - NO READING
 9-10 0.23736
 10-11 0.20722
 11-12 0.32290
 12-13 0.22386
 13-14 0.43989
 14-15 0.21554
 15-16 0.21062
 16-17 0.21474
 17-18 0.17596
 18-19 0.96746
 19-20 0.09502
 20-21 0.26987
 21-22 0.24343
 22-23 0.94333
 23-24 0.20224
 24-25 ~~0.12923~~ 0.12923
 2

BM 1 EAST

1-3 NO READING TOO FAR APART
 2-4 0.40576
 3-5 0.11745
 4-6 0.22257
 5-7 0.3232
 7-8 NO READING - TOO CLOSE
 8-9 0.08117
 9-10 0.40484
 10-11 0.04627
 11-12 0.29426
 12-13 0.29763
 13-14 0.12192
 14-15 0.43953
 15-16 0.16933
 16-17 0.42976
 17-18 0.23000
 18-19 0.4218
 19-20 0.21482
 20-21 0.39741
 21-22 0.15416
 22-23 0.26248
 23-24 0.24623
 24-25 0.14047

BM 4 EAST

1-3 0.18078
 2-4 0.43190
 3-5 0.39188
 4-6 0.64837
 5-7 NO READING TOO FAR APART
 7-8 0.22036
 8-9 0.50637
 9-10 0.19571
 10-11 0.13524
 11-12 0.23112
 12-13 > NO READING #13 EXPOSED BACK LOGS
 13-14
 14-15 0.32085
 15-16 0.19631
 16-17 0.24535
 17-18 0.24823
 18-19 0.13790
 19-20 0.49518
 20-21 0.12203
 21-22 0.30416
 22-23 0.18436
 23-24 0.17925
 24-25 0.21664

BM 1 WEST

1-3 0.40448
 2-4 0.37350
 3-5 0.10552
 4-6 0.46448
 5-7 0.45542
 7-8 NO READING TOO FAR APART
 8-9 0.35688
 9-10 0.34378
 10-11 0.12622
 11-12 0.30307
 12-13 0.13581
 13-14 0.23223
 14-15 0.04882
 15-16 0.42259
 16-17 0.19407
 17-18 0.13531
 18-19 0.18853
 19-20 NO READING TOO CLOSE
 20-21 NO READING TOO FAR APART
 21-22 0.40567
 22-23 0.14364
 23-24 0.40473
 24-25 0.14006

BM4 WJST

1-3 0.21016
2-4 0.128543
3-5 0.166450
4-6 0.25618
5-7 0.16991
7-8 0.20633
8-9 0.16540
9-10 0.43221
10-11 0.44453
11-12 TOO FAR APART NO READINGS
12-13 > DID NOT APPLY #13 NO READINGS
13-14 >
14-15 TOO CLOSE NO READINGS
15-16 0.08782
16-17 0.41525
17-18 0.21004
18-19 0.32187
19-20 0.33791
20-21 0.18142
21-22 0.23621
22-23 0.28623
23-24 0.62797
24-25 0.36494

SIKADUR 31 HI-MOD GEL

High Modulus High Strength
Structural epoxy adhesive

MTS System

Model 505.30

Serial # 10201408

Assembly # 100-155-66B Rev A

US1.31011

Pin 100-143-630

Steve. weinberger@mts.com

Scale # 508127

31264
31325

9/17/13

Open Station Mgr

Configuration how box fits
to detector

Parameter sets tuning & calibrations

Closest basic testware

Function generator to warm up
system

Reset Interlock

Upper limit stop displ control
Detector status

HPU Low Pump High

HSM - Manifold Low/High

Manual Command

Tuning Password

Tuning

Channel 2
- Conks

Cleaned intake
- Tuning Mode

Detector - All Disabled

Turned pump on
HPU First
HSM Second

± 5 in

Carla 464403-06 LVDI
SLOT 4 - J4

Servo Valve SV
397083-06

Channel 2 Defllect

.25 Amplitude
0.5 Hz

Manual Command Enabled

Scribe - Visual Graph

Quick Visual

Station Controls

Auto Offset

Use before applying Load
and displacement

Use Auto Offset

Detectors - Physical Detectors

Display
Station Set Up

±0.25 0.15 Hz

Driver Breaks up Damushon
Servo
Set to barely near it

Timing

With Square Wave
then go back to sine wave

pg 251 in Manual

Applications

Basic Testware - Simple Program
Ramp on cycle Take Limit
Leave alone

① Use multipurpose testware

② Procedure Editor
Ramp Style, Dwell

Level Crossing Acquisition

Event - Operator event

Grab ~~Ramp~~ Procedure Set
Defaults

~~Last~~ Last Ramp back to zero

Timed Acquisitions

③ Saved as Demo Ramp

④ Open procedure

Reset Time

⑤ Create Program

⑥ Have to unlock to run data

8/21
Multipurpose Software
Specimens
File name.

Warm up with Sine wave
Generator.

Questions
MTS Cell Center
1800-328-2255
Help line
Materials Tech Support

Flux test GT
Version of Software
793. Version 3.5C

Function
Sof Site #

MTS Flux Test
Electronic Doc.

1-12/82 123.60
7/10 116.80

ZIP Tie
90 Battery
10-32 2 1/2" long 2mm
1248 Connector Cable
1540
1521

9/24
To CRISTE

Channel 1. c/g / Defect

Survey Span Actual Measure
Plastic on one side

Consider Self Wt impact of
Combines - Take out

Variable Manulatore

One person

1 3.0
 2 2.4
 3 2.4
 4 3.2

Survey of Beam

Dimension At End 3/4" CHAMFER ON LEFT

	END	10'	20'	30'
T	8 3/8	8 1/16	8 1/16	8 1/16
B	7 1/8	7 7/16	7 1/4	7 1/16
L	12 3/16	12"	12 1/8	12 1/8
R	11 1/4	11 1/4	11 3/8	11 1/4
S	3 1/16			

	40'	50' END
T	8	8 1/4
B	7 1/4	7 5/8
L	12 1/16	12 1/8
R	11 3/16	11 1/4
S		

3

28 DAY Concrete Strength
1012 1013 113

BM 1 12.712.5
 BM 2 8394.5
 BM 3 6304

C0.62-L

Survey Data For BM 3 Middle

	END (West)	MIDDLE	END (East)
T	8"	8"	7 7/8
B	7 1/8	7 1/8	7"
L	12 3/16	12 1/16	12"
R	11 3/16	11 3/16	11 1/8
Strand	3 1/4"	4" side to side	3 1/16"

Overall Length 20'-9 1/2"
 10" Bearing Pad @ West End

North Side 3" x 6" x 3" Corner broke off

LUDT for Vertical End Control
 is 2" from END

TESTING OF SPECIMAN C0.62-6
IN THE MORNING I CHECK PROCEDURES
AND FINALIZED TESTING PARAMETERS

= 23.4

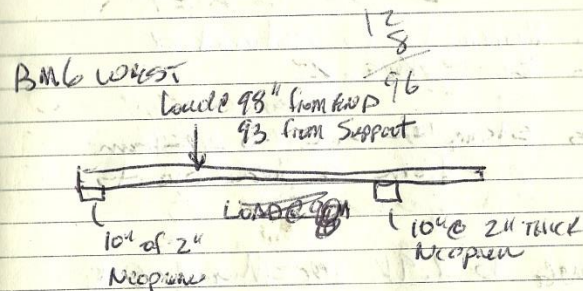
LEAVING SAMPLING RATE @ 1 sec

AN 11 FINISHED OUR PROGNOSIS TO 12,000 LB. IT WAS OBVIOUS THAT THE BEAM HAD YIELDED AND THE REINFORCEMENT WAS STRAIN HARDENED AND WAS RESPONDING PLASTICALLY (SPECIMEN C062-6). WE ADDED LOAD TO 12,800 INITIALLY AND 12,900 AT 100 LB INCREMENTS. ~~AT~~ AT 12,800 & 12,900 WE WERE GETTING CONTINUED DEFLECTION W/ AN ADDITIONAL LOAD, I.E. PLASTIC BEHAVIOR. THIS CONTINUED UNTIL THE CONCRETE AND COMPRESSURE STEEL FAILED. THE COMPRESSURE STEEL BUCKLED AND THE CONCRETE FAILED.

Only a small amount of SLP
was measured (3 0.3mm) at end
of specimen.

Survey Data For BMLWkst

	WISY	MUD	40
T	8 3/4	8 1/4	8
L	12" 12 1/2	12"	12
SURAND	3" / 4" FAS		
	3" / 4" WISY		



1/4 POINT 10'-5" From Road

1/2 point 20'-5" from RWD

3/4 POINT 301.54 from FEND

Neogene from 40' - 40' 10" from
END

Specimen length 50'-0"

LYDTs are Positioned

- 1) CW Strand to monitor stand slip
- 2) At $\frac{1}{2}$ SPAN
- 3) At $\frac{3}{4}$ SPAN
- 4) At 42' From END
- 5) At 50' @ END

10/22/13

We are set up to begin
testing of Specimen A0.7-6

Initial Cracking load is
calculated to be 1.93K

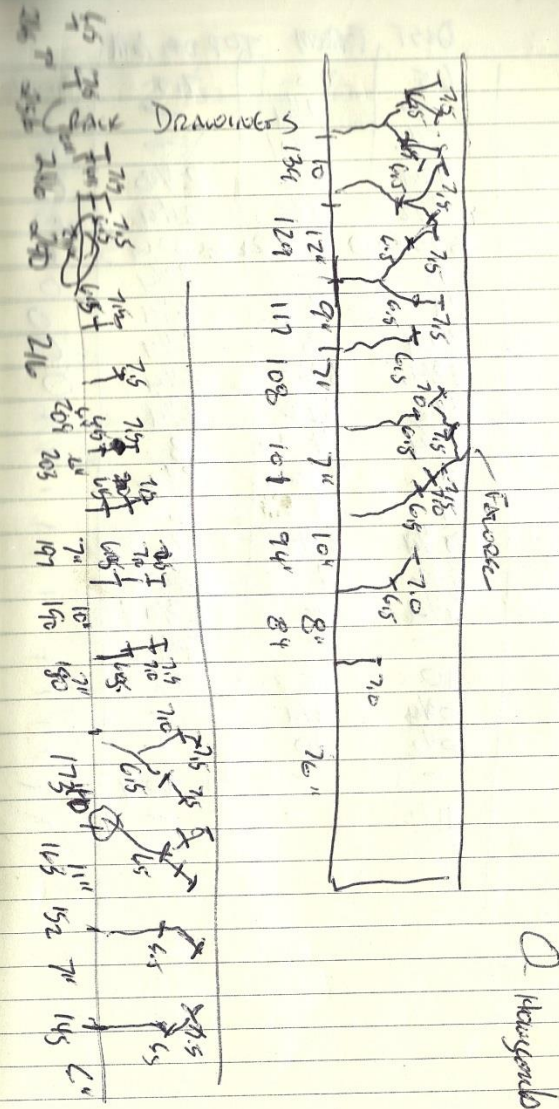
fps Monitored is estimated
to be 6.88K

fps strain ≈ 0.0125 μ /strain
from applied loads

Grown to 7.9K we Maxed out
our displacement and had to
retract actuator and add
shims.

The specimen broke at
about 7.5K w about 7" total
deflection.

Failure mode was concrete
crushing failure.



	DIST FROM TOP OF BOLL			
76	6.5	7.0	2.5	FAIRWEATHER
84	5 1/2	7 1/4	—	—
94	4 1/4	4 1/4	—	—
101	3 1/2	3 1/2	2 1/8	0
108	3 1/2	2 1/4	2 1/4	0
117	BROKEN	NO READING	—	0
129	5	—	2 3/4	0
139	4	3	2 1/4	0
145	3 3/4	3	2 1/4	0
152	4 3/4	3 3/4	2 1/4	0
163	4	3 1/2	2 1/4	—
173	5 1/4	4	3	—
180	4	3 1/2	2 1/2	—
190	9	8	6	—
197	8 3/4	6 3/4	6	—
203	9 1/2	7	6 1/4	—
209	8 1/4	—	6 3/4	—
216	—	—	8 1/4	—
240	10	8	—	—
246	10 3/4	9 1/4	—	—
256	10 1/2	10	—	—
265	—	—	10 1/4	—
	11 1/2	—	—	—

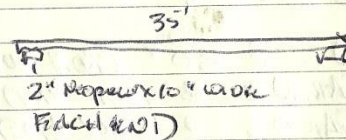
d₁ 2 1/32 | 2 1/32

d₂ 9 3/8

d₃ 10 7/16 | 10"

6/25

BIGAN TESTING SPECIMEN COIL-6
35 FT LONG



FAIRWEATHER AT 22, 450 ±
NO SLIP

WELSH LU ASSISTED

LUT'S CO 1/4 PT, 1/2 PT, 3/4 PT, 16" FROM
EAST END

Specimen B0.7-6 / BML EAST
END

Test on 10/30/13

Specimen is 24ft long
LVDTs @ #1 SLIP ON STRAND
2 1/4 POINT
3 1/2 POINT
4 3/4 POINT

STRAND GAGES
1, 2, 3 & 5 APPEAR TO
WORK

#4 IS BROKEN

LOAD POINT @ 30" FROM AND
6" BEARING @ TEST END
10" BEARING @ OTHER END

CNL CRACK 11K
FB Moment 24.9K

10/30/13

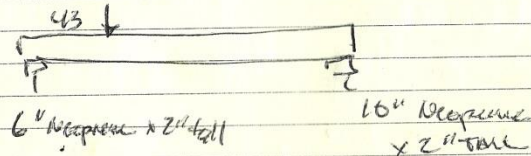
BEAM TEST w/ Yougion assisting

TEST FAILED WITH SLIP FAILURE:
BRITTLE SHEAR FAILURE AS
CRACK DEVELOPED

11/1/13

Prepared Specimen D0.70-6
for testing

LOAD POINT @ 43" FROM END



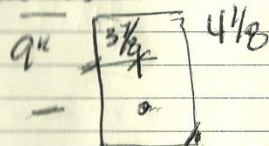
Beam is 16-11 5/8" long
The end is unreinforced w/
mild steel and is what is remaining
from the brittle failure of
Specimen B0.7-6. There are
no apparent cracks at the end of
the beam. When the first flexure
crack formed in Specimen B0.7-6
an immediate shear crack formed
and followed by a strand slip.
Further shear cracks were
not observed.

Dimensions of Beam
Specimen D-017-L

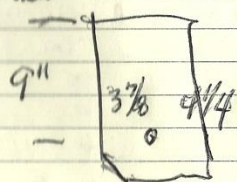
	Top	Bot	Side	Spine
Test end	8 1/8	8 1/8	11	12 1/16
Middle	8	—	11 1/8	12 1/16
Dead end	8 1/8	8	11	12 1/8

Specimen length 16'-11 5/8"
Shovel location

Dead end



Test end

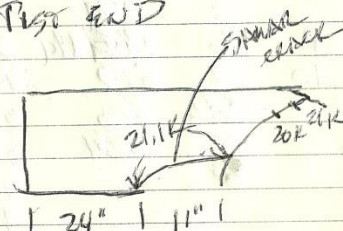


Tested beam 11 1/2 lbs
Bin Li

LVDI CH 5 IS AT SCIP AND
LVDI CH 6 IS 1/2 SPAN
CVDI CH 8 IS AT 3/4 SPAN

Specimen D-017-L
CRACK PATTERN

Test end

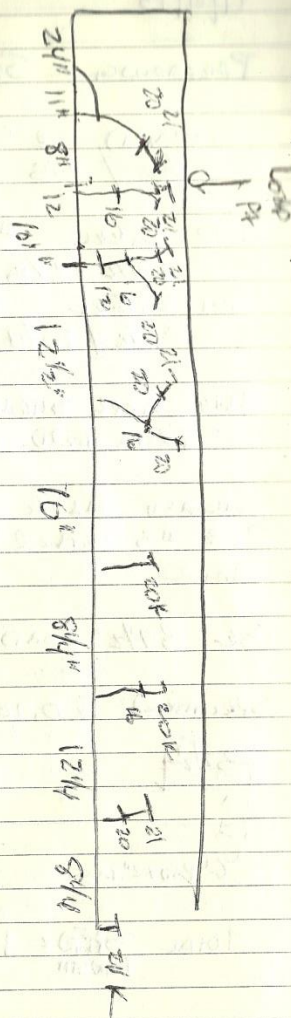


A FLEXURE CRACK FORMED AT THE TEST END OF THE BEAM AT 20K LOADING, WE TOOK OUR MEASUREMENTS AND RAISED THE LOADING TO 21K. THE FLEXURE CRACK CONTINUED TO THE 21 K POSITION AND WE MAINTAINED THIS LOADING. THE BEAM BECAME PLASTICALLY AT THIS POINT AND WE TOOK SURFACE STRAIN MEASUREMENTS. WE THEN INCREASED THE LOADING RATE DIRECTLY TO THE POINT A SHEAR CRACK FORMED WITH A SUDDEN BETWEN FAILURE OF THE SPECIMEN. SHEAR CRACK FORMED AT 21" AND EXTENDED TO FLEX CRACK @ 35"

INSERT 8 IS AT 2" FROM
 FIRST KND
 INSERT 9 AT 6" AND
 OTHERS @ 8" THEREAFTER

DIST FROM KND	21	LOAD			
		12	16	20	24
		DIST FROM TOP			
35	2	-	-	2 3/4	2 3/4
43	1 3/4	11	6	2 3/4 (4 1/4)	2 3/4
53	2	10	7	5 1/4	2 3/4 (4 1/4)
65 1/2	8"	-	6 1/2	5 1/4	5 1/4
75	-	-	-	5 1/4	5 1/4
83 1/4	-	-	8	5 3/4	5 3/4
96 1/2	8 1/4	-	-	9 3/4	9 3/4
103 1/4	9 1/4	-	-	-	-

THE STRAND IN THIS SPECIMEN
 DID NOT SLIP. FAILURE
 OCCURRED AS A COMBINATION
 OF SHEAR AND FLEXURAL
 CRACKING.



11/4/13

Preparation Specimen E0.70-6

STANDARD @ 9" FROM TOP
3 7/8" / 4 1/8" - OFF CTR BY 1/8"

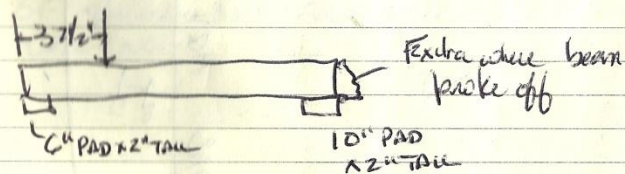
TOP STEEL #3
2 7/16" / 2 1/8" 2.1875 AU
BOT STEEL #3
9 7/8" / 10 1/4" 10.0625 AU

THE STEEL BEGAR STARTS @
6" FROM END AND AT 30"

SHOULD HAVE AT LEAST
2 OR #3 VERTICAL SHEAR IN SHEAR
PLANE.

USE 3 7/2" LOAD POINT

SPECIMEN E0.70-6



TOTAL SPAN = 158" = 13'-2"

CRACK FORM 2 9A3K f_p 30.82K

11/5/13

Yong Jun Zhou (Joe)

Started test @ 8:50 w/ Joe
assessing TOP Initial Surface
and exterior Surface measurements

TESTED Specimen to 24,000 lb
with readings @ 12, 21, 24 Kips
as we approached 26K we had a
Shear failure, which ~~caused~~

Note that this specimen end had
vertical shear reinforcement
2 ea #3 bars @ 12" O.C.

and mild steel at the top & bottom.
There were also flexure cracks
in the end zone from previous testing.

I wanted a chance to evaluate whether
this kind of cracking with
mild steel reinforcement would have
an adverse impact upon the test.

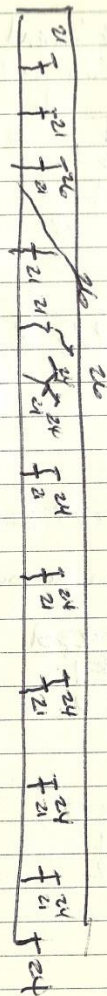
The cracks did extend from the previous
cracks within the specimen.
A few new cracks were also developed
at a place or two. Finally a
shear crack formed.

A $\frac{1}{2}$ mm strand slip was evident at a loading of 21 K. A total slippage of approximately 2 mm was observed at failure.

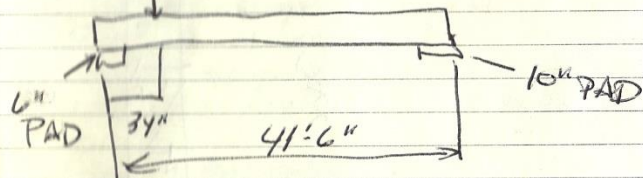
Dist from end	0	12	21	24	26
5 1/2	5	5	3 1/2	—	—
10 1/2	5 1/2	5 1/2	5	—	—
17 1/2	8	8	5 3/4	5 3/4	3
23	3	7	7	7	0

25/12	53/4	53/4	63/4	63/4	0
36	—	6 —	6	6	15/8
39	29	9	7	5 1/2	—
47	10	10	8	5	—
54	10	10	6	4	—
61	10	10	5	3	—
70	—	—	7	6	—
80	—	—	8 1/2	7	—
93	—	—	—	—	10

Place Forward EO, 10-6



31 1/2
 20-9
 10-4 1/2
 Near the test
 Specimen A0.70-8 (Beam 5 East)
 RAS SHORT-SPAN D TO 41'-6"



LVDTS @ 10'-4 1/2"
 20'-9"
 31'-1 1/2"

~~2007 K for Sp~~

Compressive Strength Tests

Specimen	PSI	lb
#1	6121.9	76930
#2	6180	77660
	12301.9	
	6150.9	

Specimen	PSI	lb
#1	7866.9	
#2	6923	
	14834	
	7417	

Estimated Responses

Cracking 7.60K
 f_{ps} 20.41K

Strain Gauge Positions

#1 @ 25"
 #2 @ 38"
 #3 @ 56"

Specimen Size

	Live End	1/2	1/2	3/4	Dead End
T	8	7 5/16	8 1/4	8	8 7/8
B	7 5/16				7 7/8
Side	12 1/4"	12 1/8	12 1/8	12 3/16	12

Strand Position 9 1/2" Live End
 9 1/4" Dead End

Point #18 is 14 1/4" from end

The specimen failed with a
 flexural failure without sup.
 BID produced a video of the
 specimen at failure. There
 was the formation of a
 rounded shear flexure crack on
 the test side. In this case
 the crack formed at 26" from
 the end of the beam.

The plastic hinge formed just to the load side of the shear reinforcement cage.

[illegible]

Dist from END	Load Point DISTANCE FROM TOP				
	15K	18.5	19.5	21	23
25	-	-	-	4	13/4
35	-	5 1/4	3 1/2	3	2 3/4
40 1/2	9 1/2	6	4 1/2	3 1/2	2 1/4
48	-	5 1/2	4 3/8	-	2 1/4
56	9	4 3/4	-	2 5/8	1 3/4
65	-	6	-	3 1/2	2
79 1/2	-	4 1/2	4 1/2	0 1/2	2 1/2
87	-	-	4	-	4 1/2
91	-	5 1/2	-	-	4 3/4
97	-	6	5	-	4
102 1/2	-	5 1/2	-	-	-
109	-	5 3/4	4 3/4	-	-
115	-	4 3/4	4 1/2	-	-
119	-	6 3/4	5	-	-
126 1/2	-	7 1/4	5 1/2	-	5
133	-	6 3/4	4 3/4	-	3 3/4
138 1/2	-	7 3/4	6	-	-
145	-	6	5	-	-
151	-	6 1/4	-	-	-
155	-	6 1/4	5 1/2	-	4 3/4
164	-	5 3/4	5 1/4	-	4 1/2
170	-	5 3/4	5 1/4	-	-
175	-	5 3/4	5	-	-
181	-	6 1/4	-	-	-
187 1/2	-	8	6 3/4	-	5 1/4
196	-	7 3/4	6	-	5 1/2
199	-	8 1/2	6 1/4	-	-
204	-	7 3/4	6	-	5 1/4
214	-	7	6	-	5

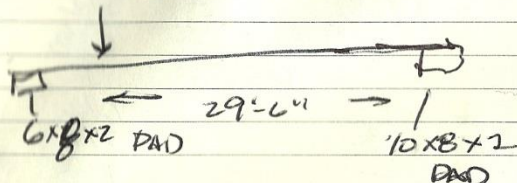
	15	18.5	19.5	21	23
223	-	7 1/2	6 1/4	-	-
228	-	6 3/4	-	-	-
235 1/2	-	9	8 1/4	-	-
245	-	7 3/4	-	-	5 1/2
251	-	9 1/4	6 1/2	-	5 3/8
258	-	9 1/4	6 1/2	-	5
265	-	10	-	-	6 1/2
272	-	8 3/4	-	-	7 1/4
276	-	-	-	-	9 3/8
284 1/2	-	-	9 5/8	-	7
295	-	-	-	-	7 1/4

Specimen B0.70-8

Beam 5 EAST

Overall length $\approx 29'-6"$

Load Point $\approx 24'$



LVDI @ 14'-9" CH2
 LVDI @ 7'-4 1/2" CH3
 LVDI @ 22'-1 1/2" CH4
 LVDI @ LINKS STRAND END CH5

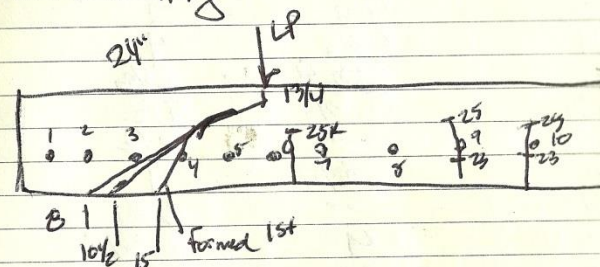
BEAM MEASUREMENTS

	END END	MIDDLE	DEAD
T	8	8	7 15/16
S1	12	12	12
S2		11 7/8	11 7/8
STRAND	9 1/4	-	9 1/2
B	7 7/8		7 15/16

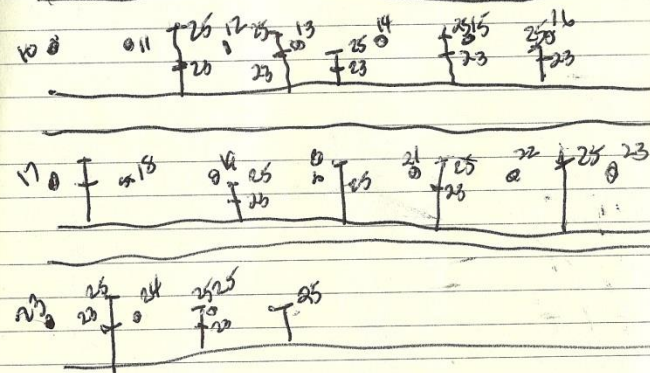
Tested Specimen B0.70-8
 on 11/24/13.

We had a shear failure at a load of 27.854 lb accompanied by a slow strand slip failure.

Upon increasing the displacement the slipping strand would hold 15,071 lb while slipping.



Dist @ Strand 12, 15, 16 1/2



CRACK
DIST FROM

DIST FROM TOP

Dist from	232	251
25	-	5 1/2
31	-	10 1/2
44	10 1/4	7
52	10 1/4	6 3/4
63	10	7 1/4
74	9	7 3/8
81	10 1/2	8 3/4
90 1/2	9 1/4	7
99	9 3/4	9
111	10 3/4 8	6 3/8
126	10 3/4	9 1/2
133 1/2	-	8 1/2
141	8 1/4	9
154	-	6 3/4
162	9	6 3/4
168	10	8
177	-	9 1/2

14
12
140
16
152
12
164

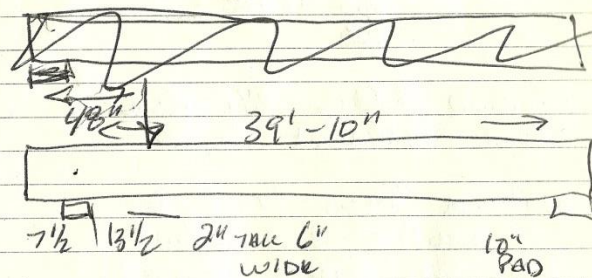
12/13

Proposed Specimen BM2 west

AD162-B (7500 psi)

Embedment Length to Load Point
= 48"

LUDTS @	END	CLAMPING
9'-9"	9'-6"	7
18'-9"	19'-9"	6
24'-	29'-10"	9



Rotational @ 26"

END	L102K	W102K	END
T	8 3/8	8	8 1/16
B	8"	7 1/8	7 1/8
H1	12"	12	12
H2	11 1/4	11 1/4	11 1/4
S	9 1/2	9 1/2	9 3/4
	4 1/4	3 1/4	3 1/4

12/16/13
 Specimen AD-62-B is 39'-10"
 Load Point 48"
 Live and Potable Point
 @ 26"

Dead W.D. @ 24"

Calc Capacity 11.5K 5.2K
 Calc Fps = 25.10K 14.1K

Staging @	6K	16
	8.5	10K
	10.0	12K
	12.5	14.5K
	15.0	

Position #1 is 8 1/4' From
 Strand Embedment Position

Noticed first crack at
 12,000 lbs @ Load Point
 No other cracks observed.

Finished 14.5 and increased
 deflection to 15.9K. Add this
 displacement and force right to 15.0K
 for strain measurements at
 that point

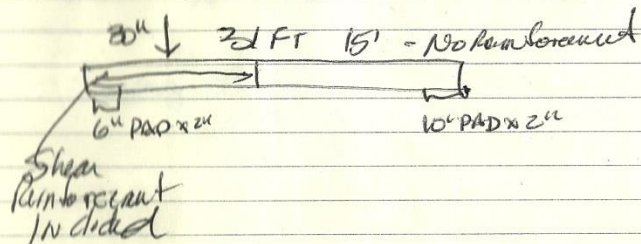
We hit the plastic limit of the
 strand the load point was
 near the transition of the
 unreinforced and the reinforced
 section. No flexural crack formed.
 I decided to cut the end
 off and retest this end with
 full reinforcement at a shorter
 distance ~ 30"

12/31/13

SPECIMEN C0.62-8

TESTED THIS SPECIMEN AFTER
A SPECIMEN WHICH BROKE AT
STIFFNESS POINT - OF 48" IN
FLATURAL FAILURE W/O SUPPORT

CUT OFF "R" SIDE AND TESTED AGAIN



LIDTS @ 7'-9", 15'-6", 23'-3"
LOAD PT @ 1'-30"

LOOK SIDE

TOP STRUT 2#3's @ 23/8"
BOTTOM STRUT @ 2#3's @ 10/16"
STRAND @ 9/16 4/8 / 3/8

Specimen Size
8" x 12"

Reinforcement Point
Lower End = 12"
Dead end = 18"

Calculated Cracking Force = 10.2 K
Calculated Fps Max Force = 28.8 K

LIDTS @ 7'-9" CHANNEL 7
LIDTS @ 15'-6" CHANNEL 6
LIDTS @ 23'-3" CHANNEL 8

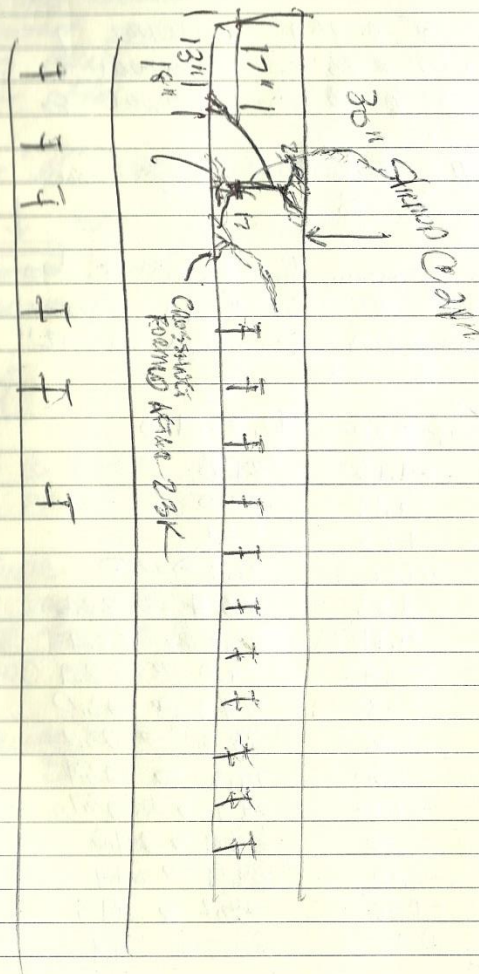
11/11 Began Testing Specimen
C0.62-8

This specimen failed with a bond
failure @ 23.9 K when the
shear crack occurred at 12"
from the end.

Specimen B0.10-10

-1.65	21.8
-1.64	22.0
-1.63	22.1
-1.62	22.8 → 22.0
-1.53	23.4 → 22.6
-1.51	23.1 → 22.7
-1.45	24.0 → 22.9
-1.40	24.0 → 23.0
-1.35	24.5 → 23.2
-1.30	24.8 → 23.4
-1.20	24.9 → 23.7
-1.10	24.9 → 24.0
-1.00	24.9 → 24.1
-0.85	25.2 → 24.3
-0.75	25.2 → 24.4

Specimen Coring Cracking
Reformation

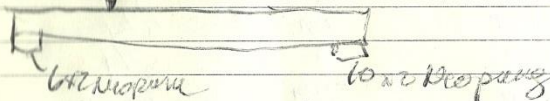


POSITION	Dist from TD		Failure
	17K	23K	
24"	8"	2"	
24.5"	6"	3"	
31-0"	7 3/4	5 1/4	
31-6"	7	5 1/4	
31-11"	7 1/2	6	
41-6"	8 1/4	5 1/2	
41-11"	8 1/4	5 1/4	
51-5"	8 1/4	4 3/4	
51-11"	7 3/4	6	
61-6"	8	6 1/4	
71-0"	7 1/2	5 3/4	
71-6"	Horizontal Coring	6 1/4	
81-0"	8	7	
81-6"	Not seen	6 1/4	
91-0"	5 1/2	7 3/4	
91-6"	5 1/2	7	
101-3 1/4"	5 1/2	6 1/4	
111-7"	8 3/4	6 1/4	
121-2"	9 1/2	8	

Specimen BD.62-8
1/24/94

Specimen Specs

40"



Span = 21'-5 1/2"

$$\begin{array}{r} 5.4 \\ 4 \overline{) 21.5} \quad 4.8 \\ \underline{20} \quad \quad \quad \text{use } 5'-5" \\ 1.5 \end{array}$$

LUDTS @ 5'-5"
@ 16'-10"
@ 16'-3"

CHANNEL 6 10'-16"
CHANNEL 7 LIVE END OF BEAM
CHANNEL 8 5'-5"
CHANNEL 9 16'-3"

BM Measurements

LIVE END

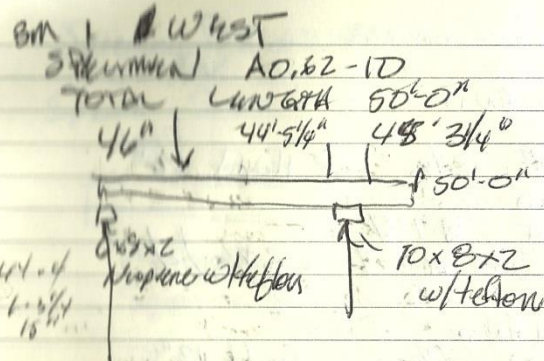
TOP 8 1/8 height 12"
MID 8
BOT 8 7/8
Strand @ 9" 3 7/8 / 4"

Dead End

TOP 7 1/8 height 12"
BOT 7 1/8

Strand @ 9" 3 7/8 / 4"

When testing this specimen, we got a slip failure @ 12.9K loading. I thought this to be a little premature. A single crack developed underneath the load point. We passed the load and had continued deflection and slippage but while maintaining 12.9 K, the final deflection @ load point was ± 0.8 in with a beginning ~~± 0.8~~ ± - 3.0 in. we stopped the test at this point.



LUTS 8' 11" LIVE END

LUT @ 11'-4"

@ 22'-8"

@ 34'-0"

LOAD POINT @ 46"

Live END

TOP 8"

BOF 8"

Strand @ 4" from Side

Regd Load for $f_{ps} = 13.2K$

6K

8

10

12

13.5

Repaired First Quarter Point
 LUT @ 11.0K

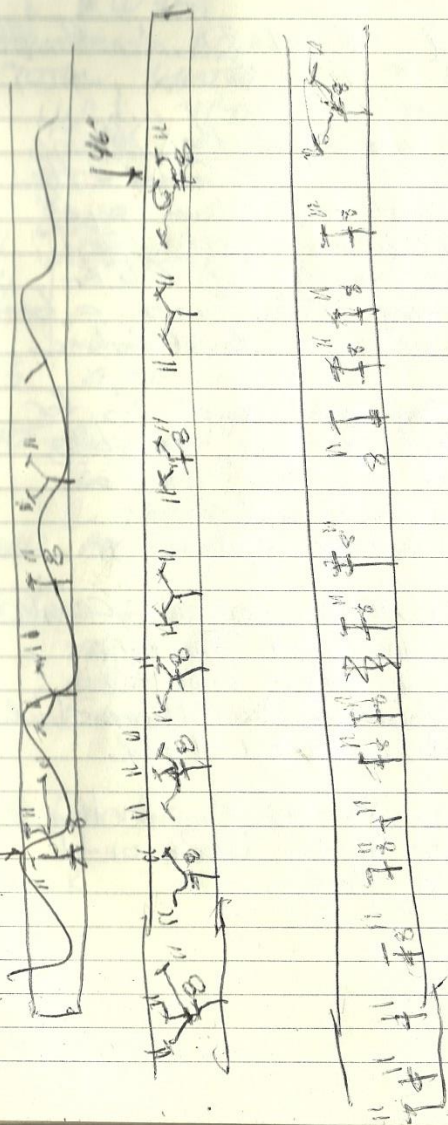
I have examined the strains from this test and found @ the load point 2% strain and at or above yield strain with a loading of 11.0K. The specimen failed with a loading of 12.8K as a bond failure.

f_{ps} strain was exceed. So for the next test we are going to repeat the test using a 46" embedment length and a shorter specimen.

I did notice concrete crushing at the load point. The beam failed as before until the bond failed. Once flexure failure occurred the strand slipped.

There was no cracking in the live end of the beam until the load point.

Crack Map 6 1062-10

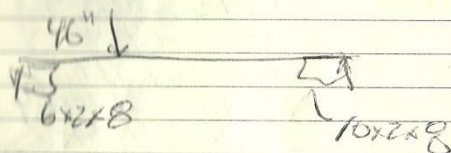


DIST FROM ROAD	DIST from TOD	
LOAD	8K	11K
46	8	4 1/2 2 1/2
69	—	4 1/4 4 1/4
82 1/2	7 1/4	3 1/2 3
95	—	4 1/4
106 1/2	6	4 1/4
123	5 1/4	3 1/2 2 1/2 4
135	7 1/2	4 1/3 1/2
154	5	4 1/3 3 1/2
168	6 1/2	3 1/4 4
177	7 1/2	3
184 1/2	10	5 1/2
190	8 3/4	4 1/2
195	9 1/2	5 3/4
206	5 1/4	4 1/2
213	7 1/2	5 1/4
221	7	4 1/2
227	7 1/2	5 1/4
237	8 3/4	6 3/4
254	—	8 1/4
262	5 1/2	6 1/2
273	8 3/4	7 1/2
285	—	7 1/4
294	—	6

Crossing @ load point @ failure
of 12.8K

Specimen B0.62-10

Length 29'04"
Load Point 46"



WTS @	CHARGE
LWR END	7
11-3	7
12-6	7
18-9	7

LWR END

100 8 1/4
100 8
HT 12
STRAND 9/4 4/4

6-15

6
8
10
12
14
15

We tested this specimen and got a shear flexural failure. Potatoes called it textbook. Just after this failure the strand began to slip from embedment distance. This is the same failure that occurred with specimen A0.62-10

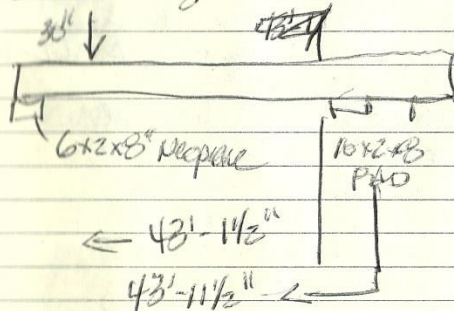
So this test is deemed repeatable.

1/21/14

Set up to test Specimen AD-70-16

Load Point @ 30" BM4 WEST

Overall length 50'



Load Positions

	Channel
Load End	7
@ 11 FT	6
@ 22 FT	3
@ 33 FT	1

Load End

Top	8
Bot	8
AT	12
Strand	9" 4/4

Set Initial Parameters for
slip loading at

9K, 13K, 17K, 20K → Deflection
afterward

Stop @ 21K if achieved

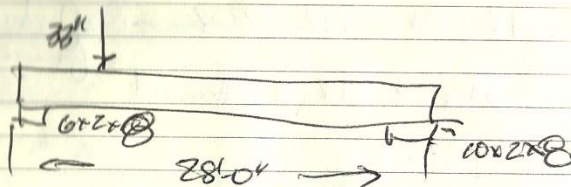
We ran the test to 20K
and took measurements, for
some reason the MTS was to
go to a displacement mode
but it failed to do that.
I had to reprogram the
the MTS under a different
specimen name and using displacement
we attained 20.168 K. We had
0.21" of slip from 20K to
20.168 K. The diagonal crack formed
at 20K and after it formed
it appears that the strand slipped
0.2". This 0.2" of slip appears
to be found mostly in the
diagonal crack area.

I would categorize this as a
slip failure near the Max limit.

20.168 K did look like it was a
plastic limit from the plot

1/30/44

Setup for Specimen B0.70-10



LUDTS

LINE END		CHANNEL #
1/4 PT	7'	7
1/2 PT	14'	16
3/4 PT	21'	9

KNO SURVEY

LINE END	
TOP	B ¹¹
101	B ¹¹
11	12.4

14	32
2	35

5	15
4	18
14	21
7	22.5

Continuing from previous page

-0.700	24.5
0.65	26.2 → 24.8
-0.60	24.8
-0.55	24.9

I made a mistake on the slip measurement @ 24.0
@ 24.7 and = 2.0660

-0.45	24.5
-0.35	24.
-0.30	
-0.15	25.0 25.2

Box Measurements @ 0.15
Had Nice Floor, Flex greater than yield stress.

23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100
23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100

24

796

7

[illegible][illegible]

Spanner 6010-10

DIST	18	21	22.3	24.7
	DIAGONAL CRACK			
26 1/2	—	5	2 1/2	1 1/2
36 1/2	6	4 1/2	4	2 1/2
48 1/2	6 1/4	3 3/4	4	1 1/2
58	5 3/4	4	3 1/2	2
64	5 3/4	4	—	3 3/4
74	6	4 3/4	3 3/4	2 3/4
83	6 1/2	4 3/4	—	2 1/2
96	7	5 1/2	5	4
107	7 1/4	5 1/2	4 1/4	3 1/4
120	8	5 1/2	4 3/4	—
129	8	6	—	4
137	8	6 1/2	5 1/4	3 1/2
146	7 3/4	6 1/2	—	6
158	—	—	—	6
168	10	9	—	7 1/2
184	—	8 1/2	—	—
192	—	—	—	11

5/6/14

Finished Formwork of 1st
Composite Deck on 2/4/14
Will Pour Concrete TODAY

Camber of Beam Measurements		Dimension TOP OF	
From END	Strong Side	Form	Weak Side
20	5"	42	15/8
2	4 13/32	44	2 1/32
4	3 13/16	46	2 13/32
6	3 3/8	48	2 13/16
8	2 31/32	50	3 5/32
10	2 21/32	52	3 17/32
12	2 3/16	54	4 5/32
14	1 27/32	56	4 25/32
16	1 21/32	58	Weak Side
18	1 5/16		
20	1 1/4		
22	1"		
24	7/8"		
26	7/8		
28	3/4		
30	3/4		
32	13/16		
34	29/32		
36	1"		
38	1 1/8		
40	1 3/8		

Position of Vertical Reinforcement

Strong Side Dist From End

ADD 186	ADD 186 + 186
2 1/4	12
7 1/8	23 3/8
11 1/2	36 1/2
17 3/4	47 3/4
23 5/8	57 5/8
29 3/4	71 1/2
42 3/4	83 5/8
47 1/8	96
53 1/4	107 5/8
59 5/8	119 5/8
65 1/8	131 1/2
71 1/2	143 1/2
81 3/4	158 3/8
92	161 1/4
102	173
111 1/2	186
121 3/4	17 1/4
132	155 1/8
142	193 1/2
152	200 1/2
151 3/4	209 1/2
174	217 1/2
186	225 1/2
	233 3/4

Strain Gauge Position

Strong Side - "A" Side

①	175"	11 1/2
②	3	29 3/4
③	59 5/8	59 5/8
④	83 5/8	81 3/4
⑤	147 3/4	142

90 3/4
167
98 1/2
106

167 114
249 123

Strain Gauge Position

Weak Side - "B" Side

①	6"	④ 87"
②	31"	⑤ 148 3/4
③	63 1/4	

167
225

15	103
23	113 1/8
31	119.25
38.75	129.5
47.25	139.25
55.25	148.75
63.25	
71	
79	
87	
96	

167
206
3

61 1/4
74 1/4

Concrete Pour 3/6/14

Slump 4.75"
Temp 70°F

3/11/14

Broke Concrete Cylinders
with Sulfur Cap

#1	242520 lbs	8577 psi
#2	209140 lbs	7396 psi
#3	180380	6379

22352
7451 psi

4/7/14

Broke 3 cylinders
w/CAO from concrete deck
pool

#1	271200	9591.7 psi
	235490	8328.8 psi
I think this one had problems with the sulfur cap		
	270260.5	9535

36371 Core #1

Settings for 200K Load Cell

Signal Input Range

200K lbs
- 200K lbs

1/600

For Bridge

Vcc Source Internal

Vcc Value 10V

Bridge Resistance 350

Scale Type

2 PT LINEAR

Electrical

1st Value

0

mV/V

2nd Value

1.975

Physical

1st

- 6700

lbs

2nd

192800

Acq Mode - Continuous

Read

Rate

25K

25K

Decimate
array
V1 on block
4 out of 1000

Strain Setup

Max Input Range 20m
 Min -20m
 Gauge Factor 2.125
 Gauge Resistor 350 Ω
 Int Volt ≈ 31.626 mV
 Internal
 Ver Value 2.5 - microstrain
 On Bridge I
 Load Resistance 0

LVDT

Max 15
 Min -15
 Wires
 Sensitivity 0.06234
 mV/V 0.001 V
 4 wires
 Internal
 Ver Value 3
 Ex Amp 20K

200K Spans

G - G
~~B - B~~
 W - B
 R - BW
 B - GW

1/1000000
 160 Division
 Synchronized
 channel expansion
 SCI mod 1010

Flat square wave
 clear chart

Min 0
 Max 0.001

1003

Save to one file

Overwrite

TEXT

One header only

One column only

Tabulator

abc

Allow Temples

217-
232-2572

Temple 217-317-6014
Dave Corso

START 2:15 @ 136

211 130.876

212 129.743

2121 128.493

2122 127.452

2123 126.683

2124 125.167

2125 124.434

START 2:27 16.0^k + 123.3^k = 139.3^k

2:28 16.6^k + 122.2^k = 138.8^k

2:29 17.8^k + 121.1^k = 138.9^k

2:30 19.8^k + 119.7^k = 139.5^k

2:31 21.8^k + 117.8^k = 139.6^k

2:32 23.6^k + 116.5^k = 140.1^k

2:33 25^k + 115.3^k = 140.3^k

2:34 26^k + 114.1^k = 140.1^k

2:35 27.2^k + 112.9^k = 140.1^k

2:36 28.4^k + 111.7^k = 140.1^k

2:38 @ 115.2^k

2:39 118.3^k

249^k @ 20^k

236^k @ 40^k

227^k @ 60^k

219^k @ 80^k

206^k @ 100^k

207^k @ 80^k

213^k @ 60^k

STARTED AT 270^k

219^k @ 40^k

226^k @ 20^k

232^k @ 0^k

BM LUTS

CH 5 @ Support 45'
 CH 4 @ MIDSPAN
 CH 2 @ LOAD POINT
 CH 3 @ SUPPORT

Camera
 Paper
~~File~~
 Forms
 Calculations
 Safety kit

June 16, 2014

Beam Test / 0.7DA

AS-BUILT DIMENSIONS

LIVE END SUPPORT	1/2 SPAN
6 15/16"	6 13/16"
7 5/16"	5 3/16"
4 1/4"	4 1/8"
10 11/16"	10 7/8"
7 1/8"	6 7/8"
4 3/8"	4 1/4"

DEAD END SUPPORT	DEAD END
7 7/16"	7 1/16"
6 5/8"	8 13/16"
4 1/4"	4 5/16"
10 7/8"	10 13/16"
7 1/16"	7"
4 1/4"	4 7/16"

A
B
C
D
E
F

June 16, 2014

Started at 8:30

Put up safety Barriers Sign
and make final preparations.

Held Safety Meeting at 9:30
and went over procedures

Began loading at 10:30

Make Shims for both load
points, 2 Old Actuators

The load was somewhat stable
but still lost some load
during measuring.

At second load point ~~slight~~
we experience vertical ~~load~~
slight racking. This initially
occurred at 144K then at
160K and 188K

The apparatus to measure
strand slip malfunctioned
in the first & second
point. This was
and the test continued,

no were used at
next.

We finished the day after
taking the data for the
second load position.

Finished at 4:00pm

Strain Gauge Position #1
was at the end of the beam

and 2-5 were consecutive
from there

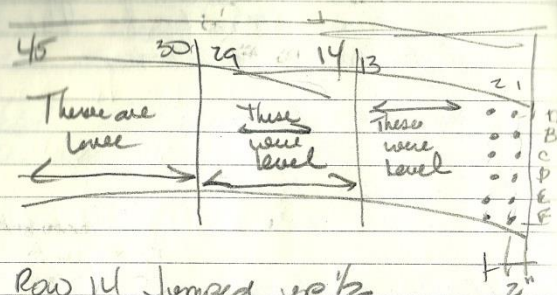
#1	12"	From END
#2	34"	
#3	58	
#4	89	
#5	148	

Experienced 1st Flexure Crack at
227K

Popped at 237K

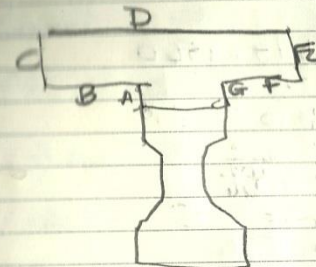
Measurement Points

110K
~~120K~~ 170K
~~205K~~ 240K



Row 14 Jumped up $\frac{1}{2}$
 Row 30 Jumped up $\frac{1}{2}$

The starting loading at Step 3 was 275 it was closer to 250K when we started measuring and found down to 200K when we finished. Average load was $\pm 225K$



LIVE END	SUPPORT	LOAD POINT	$\frac{1}{2}$ SPAN	DEAD END SUPPORT	DEAD END
A	$4\frac{3}{4}$	3"	$1\frac{1}{8}$	$2\frac{5}{8}$	$4\frac{15}{16}$
B	$12\frac{1}{4}$	$12\frac{5}{8}$	$11\frac{1}{2}$	$11\frac{5}{8}$	$11\frac{7}{8}$
C	$7\frac{1}{8}$	7"	$6\frac{3}{4}$	$7\frac{1}{16}$	$7\frac{1}{8}$
D	$36\frac{9}{16}$	$36\frac{3}{8}$	$36\frac{1}{4}$	$36\frac{5}{8}$	36
E	$7\frac{1}{4}$	$7\frac{3}{8}$	$7\frac{5}{16}$	$7\frac{1}{8}$	$6\frac{3}{4}$
F	12	$12\frac{1}{4}$	$11\frac{13}{16}$	$12\frac{1}{4}$	$11\frac{17}{16}$
G	$4\frac{1}{2}$	$2\frac{7}{8}$	$7\frac{1}{8}$	$2\frac{3}{16}$	$5\frac{1}{8}$

J.V.I. Skoffe, IL

Bearing
 10 PCI Book

Strain Gauges for embedded
 strand and O.D.B. Test

35052 GF = $2.140 \pm 0.52_0$

Bearing Pad Notes

Penny 847-675-1560

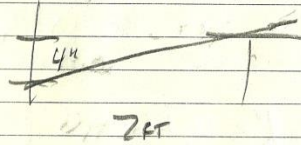
Chuck Magnesia

413-442-4147

Random Fiber Pad
3500 psi

Knox Rubber & Gasket Manufacturing

UF Folks



W/C Ratio = 0.57

5ft3 2.543
Contact = 596.5 lb/yd³ 110 lb 55
CA = 1728 lb/yd³ 320 lb 160
FA = 1320 lb/yd³ 244 lb 122
Wet = 260 lb/yd³ 48 lb 24

50.9	70.6	
53.75	72.5	48
57.8	72.8	289.5
58.05	<u>215.9</u>	
60.65	30.8	1905
58.65	28.1	
<u>57.75</u>	246.7	
<u>-19.0</u>		

1728-213-208
what does 1.711

danny @ jvi-inc.com

Transverse Shear Channel 1
LE Support Channel 2
Load Pt Channel 3
1/2 span Channel 4
DE Support Channel 5

~~2B, C, D~~ B 2 →
3 C 2 →
D 2 →
E 2 →

Tested 3 Composite deck
Cylinders

① 109956.1 psi
Load 309210

② 103261 psi
292,980

③

865-673-8877
Lift Tarek Sales

COOP 992-5518

330/EA Storing

± 10/EA

Free Sec Churn St.

522-1103

24 24 Mt S.D.

37

1727.80 - Bk 400

Appendix E – Full-Scale Data Sheets

Miscellaneous Testing Data

Specimen: 0.70A
Date Tested: 6-16-2014

Load	initial	110	185	227 K	251 K
Measured Defl Support	29 3/8	29 1/16	28 15/16		
Measured Defl Load Pt	30 7/16	29 15/16	29 3/4		
Measured Defl 1/2 Span	31 1/4	30 11/16	30 3/8		
Measured Defl Support	29 1/8	29 1/16	29 3/16		
Defl @ Beam Dead End	26 5/8	27 3/16	27 5/16		
Bearing Contact Live End					
Measured Trans Slip					
Strand Slip Measurements					
1	1.003	0.965	0.931	0.884	.805
2	1.040	1.028	1.022	1.044	1.048
3	.976	0.956	0.903	0.910	.869
4	.989	0.972	0.957	0.938	.908
5	.998	0.987	0.983	0.947	.926
6	.993	0.993	0.995	0.927	.949
7	.977	1.062	1.066	0.995	1.136
8	1.084	1.069	1.020	1.033	.988
9	1.116	1.095	1.079	1.015	.972
10	1.032	1.045	1.046	0.822	.817
11	.976	1.000	1.004	0.779	.818
12	1.011	1.048	1.050	0.888	1.035

We had to repair
plexiglass and reset our monitors
The compressor at the end moved the plexiglass

Miscellaneous Testing Data

Specimen: 0.70B
Date Tested: 6/23/14

LIVE
AND

DEAD
END

Load	INITIAL	Step 1 - 141K	Step 232K	Step 3 285	
Measured Defl Support	28'5/16	28'3/4	29'1/4	28'5/8	
Measured Defl Load Pt	29'1/4	29'1/8	28'15/16	28'1/8	
Measured Defl 1/2 Span	3'5/8	3'0'1/16	3'0'1/8	2'9'1/2	
Measured Defl Support	2'9'1/16	2'8'7/8	2'8'3/4	2'8'11/16	
Defl @ Beam Dead End	2'7'5/8	2'8'1/8	2'8'5/16	2'8'1/2	
Bearing Contact Live End					
Measured Trans Slip					
Strand Slip Measurements	- STRAIN GAUGE WITHIN CONC. BLOCK				
BOTTOM 1	446 $\mu\epsilon$	417 $\mu\epsilon$	384 $\mu\epsilon$	313	
2	BROKEN STRAIN GAUGE MIDDLE STRAND				
TOP 3	513 $\mu\epsilon$	567 $\mu\epsilon$	650 $\mu\epsilon$	-667	
4					
5			0.148		
6					
7					
8					
9					
10					
11					
12					

De 974-7704
661-8043

HAD FLEXURE CRACK AT 232K
i CRACK IN 4th strand of beam

Vita

John Verlin Cabage, Jr. was born in Hampton, Virginia, USA on November 21, 1961. He received a BS Degree in Civil Engineering from the University of Tennessee in June, 1984.

Mr. Cabage is currently pursuing a Doctorate in Civil and Environmental Engineering from the University of Tennessee. He has recently accepted a position as an Assistant Professor in Construction Management at Eastern Illinois University and begins that position in August 2014.