Technical Report Documentation Page

1. Report No. FHWA/TX-02/1852-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title And Subtitle Structural Lightweight Concrete Prestressed Girders and Panels		5. Report Date September 2001 *Revised January 2002* 6. Performing Organization Code
7. Author(s) D. B. Thatcher, J. A. Heffington, R. T. Kolozs, G. S. Sylva III, J. E. Breen, and N. H. Burns		8. Performing Organization Report No. Research Report 1852-1
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS) 11. Contract or Grant No. Research Project 0-1852
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Research Report (9/99-8/01) 14. Sponsoring Agency Code

15. Supplementary Notes

Project conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, and the Texas Department of Transportation.

16. Abstract

This report explores the uses of high performance lightweight concrete (f'c $\geqq$ 6000 psi) in pretensioned girders and deck panels.

A summary of pertinent literature on application of lightweight concrete in bridge applications is given. The results indicate a number of successful applications as well as a few cautions. The development of 6000 psi and 7500 psi mixes using locally available materials and local precasting plants is outlined. Results indicated that 7500 psi was a highly dependable maximum. Workability of the mixes was adequate, and a series of beam sections, including some purposefully highly congested members, were successfully cast under plant conditions.

When applied in pretensioned girders, the transfer length for ½-in strands was found to be 40 percent longer than the AASHTO requirement. Development lengths, moment capacities, and load-deflection curves were very similar to those of normalweight concrete girders. Lightweight concrete use in precast deck panels indicated the transfer length of smaller strands was adequate. The full calculated ultimate moment capacity of girders with lightweight concrete deck panels was developed. Such panels could be used interchangeably with normalweight concrete panels using current design requirements as long as the flexural tensile stress limits under fluid deck load are satisfied.

The report recommends that AASHTO change the model for calculating development length for prestressing strand to an expression containing the concrete Modulus of Elasticity. This would make it applicable to both normalweight and lightweight concrete. A future report will discuss economic analysis factors.

17. Key Words lightweight concrete bridges, pretensioned girders, pretensioned deck panels, transfer length, development length, lightweight concrete mix design	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.		
19. Security Classif. (of report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of pages 208	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

RESEARCH REPORT 1852-1

STRUCTURAL LIGHTWEIGHT CONCRETE PRESTRESSED GIRDERS AND PANELS

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SEPTEMBER 2001

REVISED JANUARY 2002

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Research Report 1852-1

Research Project 0-1852

PRESTRESSED STRUCTURAL LIGHTWEIGHT CONCRETE BEAMS

conducted for the

Texas Department of Transportation

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
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September 2001

Revised January 2002

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

ACKNOWLEDGEMENTS

We greatly appreciate the financial support from the Texas Department of Transportation that made this project possible. The support of the project director, Thomas Rummell (BRG), program coordinator, Michael O'Toole (BRG), and program advisor, Joe Roche (CST), is also very much appreciated. The authors also greatly appreciate the enthusiastic participation of Dr. Ramon L. Carrasquillo in the early stages of the project.

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SUMMARY

This report explores the uses of high performance lightweight concrete in pretensioned girders and deck panels. While in normalweight concrete, high performance concrete is ordinarily defined as concretes with strengths ranging from 8000 to 12000 psi, in lightweight concrete it may often refer to strengths equal or greater than 6000 psi.

The scope of this report includes four general topics. The first is a summary of pertinent literature on application of lightweight concrete in bridge applications. The results indicate a number of successful applications as well as a few cautions.

The second topic outlines in detail the development of 6000 psi and 7500 psi mixes using locally available materials and mixing practices suitable for local precasting plants. The original goal was development of 6000 psi and 8000 psi mixes but results indicated that 7500 psi was a highly dependable maximum. Workability of the mixes was very adequate and a series of beam sections, including some purposefully highly congested members, were successfully cast under plant conditions.

The third topic was the structural behavior of the pretensioned lightweight concrete girders. While the transfer length for ½-in strands was found to be 40 percent longer than the AASHTO requirement, this didn't seem to affect application. Development lengths, moment capacities, and load-deflection curves were very similar to those of normalweight concrete girders. The fourth topic was the use of lightweight concrete in precast deck panels. The transfer length of the smaller strands used in the panels was adequate. While the use of lightweight panels introduced a new mode of deck cracking, this occurred only at ultimate and the full calculated ultimate moment capacity was developed. It appears that lightweight concrete deck panels could be used interchangeably with normalweight concrete panels using current design requirements as long as the flexural tensile stress limits under fluid deck load are satisfied.

The report contains recommendations that AASHTO consider changing the model for calculating development length for prestressing strand to an expression containing the concrete Modulus of Elasticity. This would make it applicable to both types of concrete.

A future report will discuss economic analysis factors.

CHAPTER 1: INTRODUCTION AND BACKGROUND

1.1 LIGHTWEIGHT CONCRETE USE IN PRESTRESSED CONCRETE GIRDERS

A significant portion of the load carried by prestressed concrete bridge girders is the self-weight of the girders and deck. If all or part of the girder and deck can be made using lightweight concretes, there is a potential for appreciable economic savings since the self-weight could be reduced by as much as 15-20%. The availability of newer high strength lightweight concretes could make such use possible. In addition, the use of lightweight concrete can reduce transport costs and allow bulkier members to be used when crane capacity is a limiting factor. This project was undertaken to explore the pros and cons of usage of high performance lightweight concrete prestressed bridge girders and deck panels.

1.2 LIGHTWEIGHT CONCRETE MATERIAL

Concrete is composed of several components, including cement, water, admixtures, fine aggregates, and coarse aggregates. The use of lightweight coarse aggregates can lead to significant reductions in the self-weight of the concrete. Materials used as lightweight aggregate include slate, slag, palletized fly ash, and expanded clays and shales [58]. The clays and shales are mined from the ground and then placed in a kiln. As they are heated, gases are introduced and the materials expand into a hard, yet porous, material. The porosity (voids in the material) is the cause for its lighter weight relative to its volume, compared to concrete made with normal coarse aggregates consisting of gravel or crushed stone. The lightweight aggregate can weigh 40-50% less than normal coarse aggregate, and the hardened concrete using lightweight coarse aggregate can weigh 20% less than concrete cast using normalweight coarse aggregate [33]. The typical unit weight of normalweight concrete is about 145 to 150 lb/ft^3. The unit weight of lightweight concrete is usually about 120 to 125 lb/ft^3, and can range as low as 90 lb/ft^3 [58].

Many material properties of lightweight concrete compare favorably with those for normalweight concrete. The maximum compressive strength can be the same as traditional normalweight concrete, with careful proportioning of the different components in the mix design, and with careful attention to the water-cement ratio. Higher strengths may be limited by the crushing strength of the lightweight aggregate. The tensile strength, however, ranges between 80 to 100% of that of normalweight concrete. Additionally, the modulus of elasticity can be substantially lower than that of normalweight concrete. The creep and shrinkage of lightweight concrete are comparable to that of normalweight concrete, and the same multipliers and coefficients in the code can be used for both [47, 72].

The use of lightweight concrete in bridge applications has been gaining popularity since 1955, particularly in California and Norway. Bridge girder and deck applications, as well as slabs in buildings, are common uses. Lightweight concrete is particularly useful in conditions where the dead load is the major component of the loading. Further uses and applications will be discussed later in this chapter.

1.3 CONCRETE STRENGTH

Concrete used in bridge practice today generally has a compressive strength ranging from 3500 to 12000 psi. These strengths can be split into two groups, normal strength and high strength, or high performance concrete. Following subsections deal with these types in greater detail.

1.3.1 Normal Strength Concrete

Normal strength concrete is generally defined as concrete with compressive strengths up to and including 6000 psi [47]. The tensile strength of normal strength concrete is around 15% of the compressive strength [72]. Typical reinforced concrete will use a minimum of 3500 psi concrete [46], and is in widespread use in the construction industry. Prestressed concrete often uses strengths in the 6000 psi range for many applications, although recent years have seen increased use of much higher strength concretes.

1.3.2 High Performance Concrete

With normalweight concretes, high strength, or high performance concrete is ordinarily defined as concrete with strengths ranging from 8000 to 12,000 psi [33]. In lightweight concrete, high performance concrete may more often refer to strengths greater than or equal to 6000 psi. The higher strength is achieved through a mix design with a low water/cement ratio and admixtures [72]. The tensile strength of high performance concrete is an even lesser percentage of the compressive strength than normal strength concrete (around 10%). [72]

Prestressed concrete construction often involves use of high performance concrete. In fact, commercial prestressing cable anchorages used in post-tensioned prestressed construction are designed for anchoring in high performance concrete [58]. The high performance concrete has a higher modulus of elasticity than normal concrete, which results in less elastic shortening of the concrete and thereby less loss of the initial prestress force. A lower rate of creep also contributes to less reduction in the original prestress force. Additionally, high performance concrete has a higher tensile strength than normal strength concrete, which is useful in the high stress applications in which prestressed concrete is typically used.

High performance concrete is currently used in many applications, from the columns of high-rise structures to the superstructures of modern bridges [72]. The improved durability of high performance concrete due to its decreased permeability has made it particularly useful in the bridge industry, where the concrete is often exposed to harsh environments.

1.4 USE OF HIGH PERFORMANCE LIGHTWEIGHT CONCRETE

The use of high performance lightweight concrete (HPLC) is most practical in applications where the dead load of the structure comprises the structure's major loading component. This is especially true for bridge girders, as well as for the double-tees often used in parking structure designs. The lighter unit weight of the HPLC, combined with the high strength, can lead to several advantages. First, for a similarly sized section, a longer span can be employed than for a normalweight girder. Conversely, the size of the section can be reduced to span the same distance. However, since most bridge girders are constructed using AASHTO standard sections, the next smaller section may be too small to carry the desired loads. In this case, the spacing of the original girders may be increased if HPLC is employed. In some cases, the increased spacing may result in the use of one or two less girders than if normalweight concrete was used. The material savings for fewer girders must be balanced against the higher cost of the HPLC, and will vary depending on the specific job conditions [46].

An additional possible application of HPLC is in precast pretensioned deck panels. These panels comprise approximately half the thickness of the deck. They are positioned on top of the girders, spanning the spaces between them. The panels act as formwork. They are left in place when the rest of the deck is placed on top of them to form a composite member. This eliminates the difficulty and cost of getting underneath the bridge in order to remove typical wooden or metal formwork. The precast prestressed panel serves as a stay-in-place form to support cast-in-place concrete.

1.5 BACKGROUND

This report explores the use of prestressed lightweight concrete in pretensioned bridges. The area of emphasis is the behavior of prestressed lightweight concrete under load, particularly its flexural behavior. Mix designs for higher strength lightweight concrete were developed by Heffington [33]. Full details of the actual mix design of the lightweight concrete used to construct the beams, as well as a more detailed discussion of the material properties of lightweight concrete are found in the work by Heffington [33]. The transfer length of the pretensioned strands in prestressed lightweight concrete beams was evaluated by Kolozs [43]. A portion of the testing of the development length of strand in lightweight concrete beams was also reported by Kolozs [43], and was completed by Thatcher [75]. Transfer length of strands in precast deck panels was studied by Sylva [85]. Additionally, strains, deflections, strand elongation, crack patterns, cracking and ultimate loads and moments, and failure types of lightweight concrete girders are reported herein.

1.5.1 Lightweight Concrete Background

The focus of this study is the behavior of lightweight concrete beams loaded to flexural failure. This behavior is compared to the behavior of normalweight concrete beams loaded under similar conditions. Of particular interest is the actual development length of the prestressing strands in the lightweight concrete material, compared to normalweight concrete. Several equations exist in the literature for calculation of the theoretical development length. However, these expressions were developed for normalweight concrete. This study investigated the applicability of the AASHTO minimum development length equation for use with lightweight concrete.

1.5.2 Deck Panels Background

The use of normalweight concrete precast prestressed deck panels in bridge construction is widespread in Texas. Due to TxDOT's interest in the subject, deck panels using lightweight concrete were studied. The construction of the decks is treated in greater detail in Chapter 8. The key area of study of precast prestressed deck panels centers on the issue of composite action between the precast beam, precast panels, and cast-in-place deck. Barnoff performed a study on this topic and determined that full composite action was developed in cases of normalweight deck panel use [16]. The results of the current study, with respect to composite action using lightweight concrete, will be discussed in Chapter 11.

1.5.3 Development Length Background

All concrete, whether normalweight or lightweight, is strongest when used in compression. Concrete's strength in tension is significantly lower than its compressive strength. In order to use concrete in applications other than direct compression, a method of tensile resistance had to be introduced in flexural members. This led to the development of reinforced concrete, where steel bars are placed in the tension regions of the concrete member in order to resist the tensile forces after the concrete has cracked.

In pretensioned concrete members, in addition to standard reinforcing bars, high strength prestressing strands are placed in the formwork prior to casting the beams. These strands are stressed to a high tensile stress prior to casting the concrete. After the beams have been cast and the concrete allowed to gain strength, the strands are released and the tension force in the strands is transferred to the concrete as a compressive force. In order for the force-transfer to occur, a level of bond must exist between the concrete and the prestressing steel. Bond occurs over the length of the strand, and therefore a minimum length of strand must be bonded in order to fully develop the prestressing forces at transfer in a pretensioned prestressed concrete beam. This length is called the transfer length.

When a prestressed concrete beam is subject to loads, the tension zone of the concrete does not crack until the outermost fibers develop enough stress to overcome the sum of the precompression stress due to the prestress force and the tensile strength of the concrete. Beyond that level of stress, and up to the capacity of the beam, the length of bond required to fully develop the tensile resistance of the strands is called the flexural bond length. The sum of the transfer length and the flexural bond length is the development length of the beam.

Several equations for calculating the theoretical development length of normalweight concrete exist in the literature, and were reviewed by Kolozs [43]. These equations are shown in Table 1.1. This study evaluated experimentally the applicability of the AASHTO-calculated minimum development length for lightweight concrete.

Table 1.1 Development Length Equations

Author	Development Length Equation
ACI 318 / AASHTO [2,1]	$L_d = \left(f_{ps} - \frac{2}{3} f_{se} \right)$
Zia & Mostafa [18]	$L_d = L_t + 1.25(f_{pu} - f_{se})d_b$
Buckner (FHWA) [5]	$L_d = L_t + \lambda(f_{ps} - f_{se})d_b$
Mitchell [13]	$L_d = L_t + (f_{ps} - f_{se})d_b\sqrt{\frac{4.5}{f'_c}}$

1.6 OBJECTIVES

TxDOT commissioned this research project (Project 0-1852) to examine the feasibility of using HPLC in Texas bridges. Specifically, TxDOT was interested in the development of 6000-8000 psi HPLC mix designs and the performance of ½-in prestressing strands in Standard AASHTO Type I pretensioned HPLC beams. This required the exploration of several factors before a final judgment could be made on the effectiveness of the use of HPLC in this application. Therefore, this research project encompasses several different components. These components include:

Task 1. Literature Search

Task 2. Past Use of Lightweight Concrete Mix Designs

Task 3. Development of Lightweight Concrete Mix Designs

Task 4. Materials Research and Testing

Task 5. Full-Scale Testing of Type A Beams with Decks

Task 6. Evaluation of Prestress Loss

Task 7. Evaluation of Handling of Beams and Beam Behavior,

Task 8. Analytical Study of Economic Factors and Possible Benefits

Task 9. Preparation of Final Report

The overall objective of the study is to determine whether lightweight concrete is a feasible material for use in Texas highway bridges. This will be accomplished by completion of all the tasks in the research project.

The main objective of the early portion of this report (Tasks 1-4) was to determine whether high-strength high-performance lightweight concrete mix designs could be developed with $f_c^{'}$ = 6000 psi and $f_c^{'}$ = 8000 psi for use in prestressed concrete girders. The equilibrium unit weight desired was to be less than 125 pounds per cubic foot (pcf). (The equilibrium unit weight is the weight of the concrete at ambient conditions after the concrete has been allowed to shed water.)

In order to meet this objective, a variety of activities were undertaken:

a) Literature discussing high-strength lightweight concrete was located and evaluated. Of most interest was literature that investigated the use of lightweight concrete in precast bridges using an expanded clay or shale aggregate. It was hoped that this literature would give a general idea of possible mix designs along with possible expectations and limitations for the lightweight concrete and its performance.

b) Previous use of lightweight concrete in Texas was studied. Since this project was sponsored by the Texas Department of Transportation (TxDOT), it was natural to see whether TxDOT had used lightweight concrete and how well the concrete had performed in the state.

c) Two mix designs were to be developed for use in prestressed bridge girders. These mixes were to have the following characteristics:

   Both mixes should use 1/2 to 3/4 inch maximum size aggregate commercially available within the state of Texas. Also, both mixes should have a equilibrium unit weight not more than 125 pound per cubic foot (pcf).

   One mix should have a 28 or 56 day compressive strength of 6000 psi and the other should have a 28 or 56 day compressive strength of 8000 psi. Results later showed that 7500 psi was the maximum feasible. Both should achieve 3500 psi in 24 to 48 hours to permit early release in pretensioning applications. Also, the tensile behavior of both mixes should be obtained along with creep and shrinkage behavior. These tests would document the important design properties for use of the concrete mix.

d) The concretes should be workable enough for reasonable placement in pretensioned girder forms.

1.7 SCOPE OF PROJECT

The scope of the early part of this project included Tasks 1 thru 4. The research undertaken during that part was concerned with the development and refinement of two concrete mixes for use in prestressed concrete girders. To accomplish this, a total of 35 concrete mixes were developed and fabricated in the laboratory of the Construction Materials Research Group at The University of Texas at Austin.

Tests were performed on specimens of these mixes to determine the compressive strength, modulus of elasticity, and tensile strength. The unit weight was also measured. These tests provided an understanding of the behavior of these lightweight concretes.

Once the most promising concrete mixes were determined, specimens were created and tested to ascertain the creep and shrinkage behavior of the two concrete mixes used for the fabrication of precast prestressed concrete beams which were tested at Ferguson Structural Engineering Laboratory at The University of Texas at Austin.

The scope of the second part included Tasks 1, 5, 6, and 7. A literature search was performed on topics relating to the past use of lightweight concrete in structural applications. Other topics studied included transfer and development length of prestressing strand in concrete, flexural testing of lightweight prestressed concrete beams, and the use of prestressed deck panels in bridges. In addition to the literature search, the manufacture and specifications of the beams and the deck panels will also be discussed. Transfer length and development length instrumentation and testing are also included in this report. This included full scale testing of AASHTO Type I precast pretensioned prestressed concrete beams with composite slabs.

1.8 ORGANIZATION OF REPORT

This report is divided into twelve chapters.

The first chapter gives a general background of lightweight concrete as well as the reasons for performing this study. Furthermore, the objectives of the study are defined as well as the scope.

The second chapter provides a review of the pertinent literature regarding high-strength high-performance lightweight concrete. Also, reports dealing with the use of lightweight concrete in bridges are summarized.

The third chapter documents the iterative process used to arrive at the final two mixes specified for lightweight concrete with $f_c^{'}$ = 6000 psi and $f_c^{'}$ = 7500 psi strengths. The procedures used as well as the results from each portion of the study are given and discussed. Also, the thought process of how the two concrete mixes were chosen is given.

The fourth chapter documents the behavior of the 6000 psi mix. All pertinent mechanical properties are given and discussed along with the mix design.

The fifth chapter documents the behavior of the 7500 psi mix. Similar to Chapter 4, this chapter gives the pertinent mechanical properties along with the mix design.

The sixth chapter summarizes the research and gives the specifications for the two final mixes. The conclusions are presented along with possible implementation guidelines. Furthermore, recommended topics for possible future research are presented.

The seventh chapter presents background information regarding transfer and development length and gives a literature review of the material relevant to the report.

The eighth chapter includes details of the design basis for the HPLC beams and panels. A summary of the material properties of the concrete used in this project is included, and the manufacture of the components in a commercial casting yard is also discussed.

The ninth chapter discusses transfer length and gives the results of testing for this measurable quantity using two methods: concrete strain measurement and strand draw-in.

The tenth chapter discusses the development length testing set-up and test procedure.

The eleventh chapter presents test results from all twelve tests, including the six performed by Kolozs [43] and the six carried out by Thatcher [75]. The results are then compared and discussed.

The twelfth chapter of this report presents a summary of all the test results and the conclusions reached. Based on this summary, recommendations for design regulations use and application in the field are presented. Finally, areas for future study are suggested.

CHAPTER 2: LITERATURE REVIEW

To gain perspective on the use of lightweight concrete around the world, available literature from The University of Texas at Austin library as well as from a Texas Industries, Incorporated (TxI) collection of items was reviewed. This literature provided an overview of previous work done on lightweight concrete as well as its uses in prestressed bridge girders.

2.1 GENERAL RESEARCH ON HIGH-STRENGTH LIGHTWEIGHT CONCRETE

Lightweight concrete has been used for various applications in many states. However, much recent research has focused on high-performance concrete, which includes high-strength concrete. This lightweight concrete research has been undertaken to parallel progress in normalweight concrete standards of strength and workability. Also, new developments in offshore platform construction have further shaped the development and understanding of these concrete mixtures. The following discussion focuses on new developments in mechanical properties and workability aspects of high-strength lightweight concrete using expanded clays as the coarse aggregate.

2.1.1 Martinez Morales (1982) [50]

This Cornell University study looked at the mechanical properties of lightweight concrete in depth. They tested three different types of lightweight concrete, low-strength with $f_c^{'} < 4000$ psi, moderate-strength with 4000 psi < $f_c^{'}$ < 6000 psi, and high-strength with $f_c^{'} > 6000$ psi. Only results from the high-strength concrete mixes will be presented.

The concrete developed in their study utilized Type I cement and also used lightweight fines and coarse aggregate, which differed from the current study that used normalweight fines. The amount of cement was 10 sacks per yard (945 pounds), similar to the final total cementitious material for the 7500 psi concrete developed later in the current project.

For compressive strength, their concrete averaged approximately 8000 psi. The high-strength lightweight concrete also exhibited a faster strength gain than did the other varieties of concrete. 3500 psi was achieved at one day of age. The modulus of elasticity ranged from 2,500,000 to 3,000,0000 psi for all the cylinders tested.

Modulus of rupture values averaged around 800 psi for moist cured conditions and 430 psi for dry cured conditions. This showed the importance of keeping specimens wet before testing, along with the importance of moist curing on tensile strength. Splitting tensile results averaged 560 psi for wet cured and 365 psi for dry cured specimens.

Also, the authors proposed expressions based on their results for static modulus of elasticity, modulus of rupture, and splitting tensile strength. These differed from accepted AASHTO equations [1].

For modulus of elasticity,

Martinez $E_c = (40000\sqrt{f_c^{'}} + 1000000)(w_c / 145)^{1.5}$ **Equation 2.1**

AASHTO $E_c = 33 w_c^{1.5} \sqrt{f_c^{'}}$ **Equation 2.2**

(The AASHTO equation is AASHTO Equation 8.7.1 [1].)

For modulus of rupture,

Martinez $f_r = 6.5\sqrt{f_c^{'}}$ **Equation 2.3**

AASHTO $f_r = 6.3\sqrt{f_c^{'}}$ **Equation 2.4**

(The AASHTO equation is from AASHTO 8.15.2.1.1 [1].)

For splitting tensile strength,

Martinez $f_{sp}^{'} = 5\sqrt{f_c^{'}}$ **Equation 2.5**

AASHTO $f_{sp}^{'} = 5\sqrt{f_c^{'}}$ **Equation 2.6**

(The AASHTO equation is arrived at indirectly from AASHTO 8.15.5.2.4 [1].)

2.1.2 Shideler (1957) [70]

Shideler presented one of the first comprehensive studies on lightweight concrete. He tested both normal strength and high strength concrete. The high strength concrete had $f_c^{'} > 7000$ psi. He tested for compressive strength, modulus of elasticity, creep, drying shrinkage, bond, and flexural strength. Eight lightweight aggregates were used in the testing.

Shideler found he could produce concrete with $f_c^{'} > 8000$ psi using an expanded clay. He was able to exceed 3500 psi at 2 days using this aggregate. Also, he found the modulus of elasticity to be between 2,000,000 psi and 3,000,000 psi for high strength concrete using expanded clay depending on whether the test specimens were wet or dry.

Modulus of rupture was 600 psi at 28 days for the expanded clay aggregate. He also found that creep of the various lightweight concrete was greater than creep for comparable normalweight concrete.

Overall, Shideler found that performance of the lightweight concrete was good and structural grade concrete could be produced with each of the aggregates he tested.

2.1.3 Zhang and Gjørv (1993) [81]

Lightweight aggregate has often been used in Norway in offshore oil platforms. Zhang and Gjørv studied some of this lightweight concrete.

They developed nine lightweight concrete mixes using silica fume as a pozzolanic admixture. The worst performing concrete achieved a compressive strength of 8310 psi at 28 days. All mixes were 6000 psi by 3 days.

Zhang and Gjørv hypothesized that the lightweight aggregate strength would control the maximum strength of the mix. The cement content, silica fume, and sand have lesser effects.

2.1.4 Burg, Cichanski, and Hoff (1998) [23]

Since lightweight concrete has often been used in offshore oil platforms, high-strength lightweight mixes have been developed. Burg, et al were able to develop one using just cement and fly ash as the cementitious material.

The mix contained 700 lbs of cement and 200 lbs of fly ash per cubic yard of concrete. The fine aggregate was natural sand. The mix achieved a strength of 8500 psi at 90 days. At three days, the concrete had an approximate strength of 6800 psi. Although it is not noted explicitly in the paper, the concrete apparently achieved a strength above 3500 psi at one day.

The concrete had a modulus of elasticity of 4,000,000-4,500,000 psi at 90 days. The authors evaluated both Equations 2.1 and 2.2 with their data and verified that Equation 2.1 was a better fit for the modulus of elasticity data.

The splitting strength was between 250 and 500 psi for dry curing and 500-700 psi for moist curing, which exceeded values predicted by Equation 2.4.

The permeability of the concrete was rated as moderate. When the authors compared their results to a typical value from normalweight concrete, the permeability was nearly the same. The authors concluded that this particular mix was suitable for the arctic environment.

2.1.5 Nilsen and Aïtcen (1992) [59]

Nilsen and Aïtcen looked at the properties of high-strength concrete containing various types of aggregates. This report will focus on the results for concrete with lightweight aggregate.

The lightweight concrete was made with expanded shale coarse aggregate and natural sand fine aggregate. Silica fume was used as an admixture. Also, Type III Portland cement was used. The two mixes produced concrete with compressive strengths of 13,100 and 10,700 psi, respectively at 28 days of age. Also, the concretes attained 8500 psi and 7000 psi at one day of age, well above the 3500 psi needed for the current project..

They found that the AASHTO code Equation 8.7.1 [1] (Equation 2.2 in this report) for lightweight concrete modulus of elasticity underestimated the modulus of elasticity, a finding that agrees with the previous research by Slate, Nilson, and Martinez.

Drying shrinkage of the lightweight concrete was similar to that of normalweight concrete.

2.1.6 Zhang and Gjørv (1991) [80]

Zhang and Gjørv produced a later paper dealing with the properties of high-strength lightweight concrete.

This paper dealt with many of the same mixes that were discussed in Section 2.1.2 but had a different focus.

The conclusions of interest were:

a) The ratio of tensile strength to compressive strength in lightweight concrete is less than the same ratio in normalweight concrete.

b) The strength of the lightweight aggregate is the primary factor controlling the strength of high-strength lightweight concrete.

2.1.7 Mircea, Ioani, Filip, and Pepenar (1994) [53]

The authors tested 260 reinforced and prestressed beams under different aggressive environments for durability. Beams were made of lightweight and of normalweight concrete and were precracked.

The beams were then placed in various environments and allowed to sit for ten years. After ten years, the beams were analyzed and loaded to failure to see if they maintained their strength.

The conclusions were that the lightweight concrete performed as well as the normalweight concrete. The density of the lightweight concrete beams decreased 2.2% while the normalweight companion beams decreased 2.0%. Also, both types of concrete increased in modulus of elasticity with the lightweight gaining 12% while the normalweight gained 25%. Compressive strength of the lightweight concrete increased 17-25% while the normalweight gained 7-15%. Overall, the results were similar with neither concrete performing poorly.

Also, higher cement contents generally proved to reduce the size of the cracking inside the beams. Since high cement contents generally portend higher strength concrete, this means that the higher strength beams were better able to resist crack growth.

2.1.8 Reichard (1967) [64]

Reichard published one of the first studies on creep and shrinkage of lightweight aggregate concrete. His work is still the basis for the lightweight concrete creep and shrinkage recommendation by ACI Committee 213 [2].

Reichard found that shrinkage of lightweight aggregate concrete ranged from 0.02% to 0.08% of the total length at 90 days. The average was approximately 0.05%. At 2 years, the shrinkage ranged from 0.04% to 0.09% with an average approximately 0.07%. Lightweight concrete generally plateaued around 150 days of age. Very little drying shrinkage would occur after this time period.

Reichard also tested creep. It ranged from 0.06% to 0.14% of the total length at 90 days. At 2 years, the creep ranged from 0.09% to 0.22%. The average at 2 years was approximately 0.16% of the total length.

Reichard also showed that creep plus shrinkage increased as cement content was increased. It was approximately linear, with the creep plus shrinkage equaling 0.28% of the total length at 1 year for cement contents of 700 pounds per cubic yard, a similar amount to that expected in the current project.

2.2 PERFORMANCE OF LIGHTWEIGHT CONCRETE IN PRESTRESSED MEMBERS

Lightweight concrete has been used in bridges around the world. Different parts of the bridge structure have been fabricated with lightweight concrete. Results have been mixed.

2.2.1 Lightweight Aggregate Bridge Construction and Performance in Europe

European countries and especially Norway have built many bridges with lightweight concrete. They have had success with the material. Following are some examples.

2.2.1.1 Mays and Barnes (1991) [51]

Mays and Barnes looked at the performance of many lightweight concrete structures in the United Kingdom. Of most interest is their discussion of lightweight concrete bridge structures in place.

Overall, the structures were all in good shape. They showed some wear and tear, but when compared to adjacent normalweight concrete structures built at about the same time, the lightweight concrete structures actually outperformed the normalweight concrete structures. Also, measured chloride levels in the lightweight concrete were lower than in normalweight concrete structures. The performance was satisfactory for all the bridge structures.

2.2.1.2 Laamanen (1993) [45]

Laamanen discusses the Sundbru bridge in Eidsvoll, Norway which used high-strength lightweight concrete. The bridge, built in 1991-1992, utilized natural sand and lightweight aggregate Leca, an expanded clay.

Overall performance of the concrete in the bridge was excellent. The compressive strength of the concrete averaged 9700 psi at 28 days, achieved with the use of silica fume as an admixture. The modulus of elasticity was 3,080,000 psi at 28 days. The equilibrium unit weight of the concrete averaged between 115 pounds per cubic foot (pcf) and 118 pcf.

Measured chloride and freeze-thaw resistance indicated that the lightweight concrete performed as well as comparable normalweight concrete. Overall, the performance of this bridge was a success.

2.2.1.3 Melby, Jordet, and Hansvold (1993) [52]

In 1988, Norway introduced a new standard for design of concrete structures with higher limits for concrete strength. This new standard encouraged designers to use higher strength concrete in their structures.

Since high-strength lightweight concrete had become a viable option due to the introduction of water-reducing admixtures and silica fumes, designers chose it for two bridges in Norway, Sandhornøya and Støvset.

Both bridges were long-span cantilever bridges. Sandhornøya had a midspan of 505 ft and was the first bridge in Norway where lightweight concrete was used in the superstructure.

The concrete performed satisfactorily. The strength was adequate (no exact values given) while the modulus of elasticity was 3,260,000 psi at 28 days, larger than usual for lightweight concrete. After five years, the concrete was inspected for its performance. The structural state of the bridge was good with some cracking. It was theorized by the authors that the cracking was caused by the inferior curing conditions faced by the bridge. Specifically, the bridge was cured in low temperatures in the middle of the winter. Also, the concrete proved to be very resistant to chloride penetration.

Overall, the lightweight concrete proved to be economical for use in long-span bridges. The author concluded that as long as steps are taken to monitor the bridge, since durability of lightweight concrete is not fully understood, then lightweight concrete makes a good choice for a bridge material.

2.2.1.4 Sandvik (1993) [67]

Sandvik provided an overview of bridges built in Norway with lightweight concrete since 1987. Eight bridges had been constructed using high-strength lightweight aggregate concrete. All are found in marine environments. Some of the bridges included in his study are also found in the previous studies cited.

Overall, Sandvik found the use of high-strength lightweight concrete to be minimal due to the unfamiliarity of designers with the material. However, in those bridges where it was used, the performance has been comparable to that of the normalweight concrete with no major problems reported with any of the bridges.

2.2.2 Lightweight Concrete Bridge Performance in United States

Lightweight concrete has been widely used in bridges in the United States since the 1960s. Most experiences have been good as lightweight concrete has performed similarly to normalweight concrete.

2.2.2.1 Hanson [32]

Hanson wrote an early paper discussing the use of lightweight concrete for prestressed concrete construction. He focused on the expanded shale aggregate that was available in the Rocky Mountain area.

The main advantages of lightweight concrete, Hanson concluded, were the ability to produce smaller sections due to the decrease in weight of the concrete. Also, another advantage was the decreased transportation cost, as a lower weight will allow more units to be placed on a truck for transfer.

However, substantial attention was focused on the strength of the concrete. Due to the desire of precast manufacturers to release their forms in one day, a concrete mix must be developed which has sufficient one day release strength. Also, Hanson suggests that a lightweight concrete mix must also have an adequate modulus of elasticity, as this will help reduce camber of the unit, a significant problem with lightweight concrete prestressed members.

2.2.2.2 Jennings and Brewer, Florida Department of Transportation (FDOT,1964) [39]

One of the first documented experiences with lightweight concrete in the United States is from FDOT. FDOT faced a problem in that it wanted to replace a steel truss bridge that spanned 120 feet. They wanted to continue to use the same span length but to replace the structure with prestressed concrete. At that time, the 120-foot span was considered to be too long for typical normalweight concrete prestressed girder construction. Therefore, it was decided to try lightweight concrete for the substructure, superstructure, and deck.

For the bridge, the girders chosen were American Association of State Highway Transportation Officials (AASHTO) Type IV girders. Six girders supported each span of a 28 foot wide deck.

The lightweight aggregate used was Solite, an expanded clay. The specification for the lightweight concrete was that it had to have an equilibrium unit weight less than 120 pcf. The concrete performed well above minimum standards. The prestressed girder concrete tested at 6500 psi at 28 days. Although release strengths are not mentioned, it is noted that the concrete checked out well above the minimum design strengths. The deck concrete tested at 4000 psi at 7 days and 5000 psi at 28 days.

The biggest problem encountered during the construction of this bridge was the variation in moisture condition of the coarse aggregate. Florida officials chose to handle this problem by sprinkling the stockpiled aggregate for 24 hours prior to production of the concrete.

2.2.2.3 Murillo, Thomas, and Smith (1994) [56]

Another advantage of lightweight concrete for segmental bridges is in the seismic area. Lightweight concrete can alleviate two problems faced by normalweight segmental concrete bridges; the lateral forces induced by ground motions which shake the foundations of elevated superstructures and the out-of-phase oscillations of the superstructure.

Their paper discussed the choice of lightweight concrete for a 1.2 mile long bridge located in California between the cities of Benicia and Martinez. The bridge has been designed to withstand a 7.3 magnitude earthquake on the Richter scale.

The lightweight concrete box girder bridge turned out to be the most economical bridge of the four surveyed, costing $8 to $42 million less than the others. The concrete chosen had natural sand as the fine aggregate and an expanded shale as the coarse aggregate. The spans were 528 feet in the center and 335 feet on the ends.

Increasing the prestress placed into the girders, thereby increasing the camber, combatted the reduced modulus of elasticity of the lightweight concrete. Also, the box girders are prestressed longitudinally, transversely, and vertically. This three-dimensional prestressing provided for a relatively crack-free structure.

Overall performance of the bridge was expected to be more than adequate, providing increased seismic resistance for a smaller cost.

2.2.2.4 Vaysburd (1996) [78]

In his article in Concrete International, Vaysburd reported a study of durability of lightweight concrete structures. By comparing the mechanical properties of lightweight concrete to normalweight concrete, he found that lightweight concrete actually should perform better than normalweight concrete in resisting crack formation.

Vaysburd found that the lower modulus of elasticity, higher drying shrinkage, and and higher creep values of lightweight concrete compared to normalweight concrete gave lightweight concrete the ability to sustain greater tensile strains. Because of this, the lightweight concrete actually should have more crack resistance. Also, tests have shown that lightweight concrete has lower permeability values than comparable normalweight concrete.

Furthermore, lightweight concrete generally has more cement per cubic yard than normalweight concrete. Therefore, this delays the carbonation and steel depassivation (the start of corrosion) by having more calcium hydroxide available.

To back these findings, Vaysburd looked at two bridges that used lightweight concrete in their decks in the United States. The first example, the William Preston Lane, Jr. Memorial Bridge in Maryland was constructed in 1952 with an expanded shale deck. An inspection in 1975 showed that the lightweight concrete had outperformed the normalweight concrete in the bridge. Also, The San Francisco-Oakland Bay Bridge was constructed in 1936 with an expanded shale deck while the lower deck of the bridge was reconstructed with an expanded shale deck in the early 1960s. The lightweight decks showed some chloride contamination in the top inch of the exposed surfaces. However, the chloride levels at the steel layer had not reached a worrisome level. On the other hand, the parts of the bridge using normalweight concrete were in need of replacement due to spalling.

2.2.3 Lightweight Concrete Bridge Performance in Texas

From internal information provided by the Texas Department of Transportation (TxDOT), use of lightweight concrete in Texas bridges has been fairly minimal. Most, if not all, of the experience with lightweight concrete has been limited to use in decks.

Typical of the use of lightweight concrete is its use in the Rainbow Bridge over the Neches River. The width of the deck needed to be expanded to meet specifications. However, engineers did not want to increase the dead load on the structure. Therefore, lightweight concrete was chosen since it allowed engineers to obtain the increased width of the deck without increasing dead load.

Overall, performance of lightweight concrete has been comparable to that of normalweight concrete. Most of the elements constructed of lightweight concrete are rated at 6 or 7 on the BRINSAP scale, meaning satisfactory performance with some signs of wear.

Perhaps the worst performance came in the Pierce Elevated in Houston. Lightweight concrete was used in the deck and had terrible performance. There were large problems with spalling and cracking of the deck. However, it has been speculated that these problems with performance were largely caused by bad construction practices. Investigations showed that the concrete was constructed without the minimum cover needed for protection of the steel bars from corrosion. Therefore, the bars corroded and spalled, cracking up the concrete.

Otherwise, overall performance of lightweight concrete in Texas has been good. Whenever suitable construction practices have been followed, lightweight concrete has proved to be an appropriate choice of material.

CHAPTER 3: MIX DESIGNS

3.1 FIRST ITERATION

In the original goals for this project, a 6000 psi and an 8000 psi mix design were envisioned. As well be shown later the dependable design strength of the 8000 psi concrete was reduced to 7500 psi. However, in reporting on the laboratory studies, general references will be made to the 6000 psi and 8000 psi goals. In order to produce the lightweight aggregate concrete desired for the project, many different mix designs were created and tested. The initial mix designs were chosen to investigate a wide variety of materials and proportions. Later mix designs would focus on refining specific promising mixes.

Also, these initial iterations provided an opportunity for familiarizing the staff with proper use of lightweight aggregate. Lightweight aggregate requires different preparation procedures than typical aggregates, such as crushed limestone and river gravel, due to the high moisture amounts that lightweight aggregates absorb.

3.1.1 Decisions on Materials

Concrete is comprised of four distinct components: cementitious materials (includes cement and/or pozzolonic admixtures), coarse aggregate, fine aggregate, and water. A multitude of options exist from which the materials can be chosen.

However, a couple of general rules guided the process. First, the materials had to be widely available inside the state of Texas. Precast concrete plants should be able to obtain the aggregates in a timely manner. Second, the mix designs needed to be as simple as possible. Therefore, exotic admixtures or materials that are not familiar to precasters should be avoided. Finally, the mixes needed to be easily reproducible.

3.1.1.1 Type of Cement

Since this lightweight concrete was being used in a precast plant environment, high early-strength values were necessary so that the strands could be released in approximately 24 hours. The precast plant where the beams for this project were fabricated requires that concrete be at 3500 psi before release of strands. Because of these early high-strength requirements, Type III cement was chosen. Type III cement is the typical cement used in precast plants due to its high strength gain at early ages.

Many cement manufacturers exist around Central Texas. However, only one company makes Type III cement and packages it in small enough quantities for laboratory use. Therefore, the Alamo Cement plant north of San Antonio was the source for the cement for the laboratory mix designs in this project. The brand name of the cement was Alamo Red Bag. Alamo provides much of the cement for the precast plants around Central Texas.

3.1.1.2 Type of Fine Aggregate

Since the concrete was required to have an equilibrium unit weight no more than122 pcf, this allowed the use of sand as the fine aggregate of choice. It was felt that a fine aggregate made up of lightweight materials would not provide the performance needed to reach the high-strength specifications.

The sand used in the early stages of the project was Colorado River sand from Capital Aggregates. Midway through the project, a new shipment of sand was obtained. Due to a sand shortage in the Austin area, a new supplier was located. The sand from the new supplier was also Colorado River sand, similar to the earlier type.

3.1.1.3 Type of Coarse Aggregate

Once again, availability of aggregates constrained the choices for lightweight aggregates. In the state of Texas at the present time, apparently only one company produces lightweight aggregate, Texas Industries (TxI). They produce two separate lightweight aggregates, Clodine and Streetman. Clodine is an expanded clay while Streetman is an expanded shale. Discussions with CoreSlab Industries, a precast manufacturer of double-tee members for parking garages, showed that they used Streetman for use in manufacture of double-tee members. However, use of Clodine is also widespread in manufacture of lightweight concrete and slabs.

From these two choices, Clodine was readily available from a local ready-mix concrete plant, Rainbow Industries. They were willing to provide small amounts of aggregate at any time. The aggregate had a maximum size of ¾ inch and was well-graded. Therefore, Clodine was used as the initial lightweight aggregate.

3.1.1.4 Type of Fly Ash

Fly ash is often used due to the excellent permeability characteristics of concrete incorporating fly ash. A Class C fly ash was used in all the mixes that utilized fly ash. It was also obtained from Rainbow Industries.

Class C fly ash was chosen due to its widespread availability in Texas. Also, its ability to aid in the formation of late-age strength was desirable since Type III cement generally slows in its late-age strength production compared to Type I cement.

3.1.1.5 Type of Admixtures

A major concern was the workability of the concrete. Since these specific mixes of concrete needed to be used in a precast environment, this concrete needed to have a large slump.

Generally, large slumps are achieved in concrete through the use of more water in the mix. However, more water in a mix reduces the strength. Therefore, admixtures were chosen to produce the necessary slump to cast these beams.

The admixtures needed to serve two purposes. Due to the large amount of cement expected to be required, these mixes would likely have a higher temperature than normal mixes. With an increased temperature, the concrete would experience rapid slump loss. Therefore, a retardant would be needed to slow the set times. Since the laboratory had access to Daratard-17 by Grace, this was the retardant chosen.

The second purpose was to produce the slump needed for these mix designs. Again, due to the large amount of cement, small slumps were expected. Therefore, a superplasticizer was needed to increase the slump to the target of 7 to 9 inches. ADVA Superflow was the choice due to its ready availability.

Both these admixtures are widely available throughout Texas from Grace. Many precast plants around Austin use these Grace admixtures.

3.1.2 Initial Variables

After the initial decisions about which materials would be used, proportions had to be decided. To do this, existing literature was reviewed to provide some ideas about possible proportions for high-strength mixes. Also, local precasters were contacted to determine any possible high-strength lightweight mixes that they used. A local precaster used a blended coarse aggregate with crushed limestone and lightweight aggregate [39]. However, use of this would was ruled out because the mix was too heavy.

Furthermore, most literature indicated that silica fume was a key admixture in creating high-strength concrete. However, it had been decided not to use silica fume due to its high cost and low availability compared to fly ash. Therefore, most of the first mixes were based on prior experiences. Consultation with Dr. Ramon Carrasquillo provided the mixes developed for the first part of the project.

The mix designs for all the mixes are presented in Appendix B.

3.1.2.1 Water/Cement Ratio

Although water/cement ratio does not play as large a role in strength in lightweight concrete as it does in normalweight concrete, it still is a significant quantity. Due to its widespread use in the field of concrete design and its familiarity to most people in the field, it is a convenient measure for controlling concrete strength since it usually gives a rough idea of the resultant compressive strength of the concrete.

For these initial mixes, prior experience and previous literature provided a guide to initial values of the water/cement ratio. From these, values in the range of 0.30 to 0.35 were chosen. Obviously, workability is a prime issue. Therefore, the water/cement ratios needed to be as large as possible to maximize workability and minimize use of superplasticizer. Table 3.1 presents the water/cement ratios used in the first portion of this project.

Table 3.1 Water/Cement Ratios and Cementitious Material Amounts for First Iteration

Mix Number	Water/Cement Ratio	Pounds Cementitious Material/Cubic Yard
1	0.35	600
2	0.35	600
3	0.35	600
4	0.35	600
5	0.35	600
6	0.35	600
7	0.35	600
8	0.35	600
H-1	0.32	800
H-2	0.32	800
H-3	0.32	800
H-4	0.32	800

3.1.2.2 Amount of Fly Ash

Using fly ash was not a foregone conclusion in these mixes. Prior documentation has shown that fly ash reduces early-age strength of concrete significantly. Since one of the main emphases of these concrete mixes was to obtain high early-age strength, fly ash could create a problem.

Therefore, these initial mixes were made both with fly ash and without fly ash. When fly ash was used, the proportion was chosen to be 25% replacement with fly ash by weight of cement.

3.1.2.3 Coarse Aggregate Factor

Another goal of these early mixes was to ascertain the amount of coarse aggregate that is needed to produce a workable mix and the required proportion between the sand and lightweight aggregate. In normalweight high-strength concrete made with crushed limestone or river gravel, the concrete gains a significant portion of its strength from the aggregate. However, in lightweight concrete, the aggregate does not contribute significantly to the strength. Although very weak aggregate could detract from the strength, increasing the amount of lightweight aggregate in the matrix does not effectively increase the strength or the stiffness.

Thus workability became the main concern when proportioning the coarse aggregate. The proper proportion between coarse and fine aggregate had to be found in order to give the proper finishing characteristics and adequate slump. Also, since the cement-sand mortar serves as a binder in concrete, there had to be an ample amount to hold the concrete together.

For these initial mixes, two separate proportions were chosen for the coarse aggregate and the sand. It was hoped that these two proportions would provide extreme ranges on the possible behavior. In other words, one mix would have about the maximum amount of sand (making it "sandy") that could be used before the concrete would become too sticky while the other would have the maximum amount of lightweight aggregate (making the mix "rocky" or "coarse").

3.1.3 Procedures

For production of the concrete, ASTM procedures were followed. This was done in order to have the best possible comparison between previously published data and the data in this project.

3.1.3.1 Preparation of the Aggregate

The aggregate presented the most difficulties during the mixing of the concrete. Most users of lightweight aggregate wet down the aggregate for at least 24 hours prior to placement in concrete mixer. In most precast and ready-mix concrete plants, aggregate is placed in a stockpile and then a sprinkler wets the pile for at least 24 hours. The aggregate then will be somewhere between the saturated surface dry (SSD) state and the saturated state.

Production of concrete at the laboratory presented a large problem. First, no facilities were available to allow use of a sprinkler with the stockpile of aggregate. The closest practical procedure would have been to submerge the aggregate until loading it into the mixer. However, this was not desired since the aggregate would then be too wet before placement in the mixer. Also, most literature on the subject of lightweight aggregate concrete has had the aggregate added while in a moist condition [50,80].

Because of these problems, it was decided to submerge the aggregate in tubs of water for at least 24 hours prior to mixing of the concrete. If possible, the aggregate would begin soaking in the tubs 72 hours prior to concrete mixing. Figure 3.1 shows the aggregate soaking in the tub.

Figure 3.1 Aggregate Soaking in Tub Before Drying

Approximately an hour before mixing, the aggregate was removed from the tubs and then placed on a concrete deck outside. The water not soaked up by the aggregate or clinging to the surface drained away from the aggregate with the help of the sun and wind. The aggregate was then added to the mixer in a close to saturated but surface dry condition (SSD).

3.1.3.2 Production of Concrete

Concrete was produced in accordance with ASTM Procedure C685 [13].

First, a moisture content of the sand was taken so that the water could be adjusted to account for the absorption capacity of the sand. Second, the amounts of cement, fly ash, sand, and water were weighed out. The scale had an accuracy of 0.1 pound, more than ample when dealing with the size of proportions in this project.

Third, the aggregate was weighed out. No moisture contents were taken since the ASTM test for moisture content in normalweight aggregate is considered an extremely unreliable test for lightweight aggregate. The aggregate was assumed to be close to SSD state. Fourth, the mixer was buttered with 10% of the cement weight and sand weight to reduce the losses in the mixer. Fifth, the components were placed in the mixer. Figure 3.2 shows the mixer used in this project.

Figure 3.2 Concrete Mixer

The lightweight aggregate was added first followed by the sand. The mixer was then turned for a short time to produce a good mixture of the two components. After mixing these two components, the cement and then the fly ash were added. Again, the mixer was turned a number of turns and allowed to mix all the components well.

Then, the water was added. As the mixer was rotating, half the weight of the water was added. The mixer was allowed to spin until the water was accepted by the cement and fly ash. After there was no visible free water (and the aggregate and mortar was starting to clump), the rest of the water was added slowly as the mixer was spinning. This was done to aid complete mixing of the concrete.

After the water was completely added, the mixer was spun for three minutes. The concrete was then allowed to rest for three minutes. Then, the concrete was spun for another two minutes. At the end of the two minutes, a slump test on the concrete was taken.

After the slump test was taken, superplasticizer was added in 2 fluid ounce increments until concrete with the slump desired was produced.

The concrete was then emptied into a wheelbarrow. Specimens were then prepared in accordance with ASTM standards depending on need.

3.1.4 Initial Results

To accurately document the behavior of the test specimens, the test regimen in Table 3.2 was developed.

Table 3.2 Test Regimen for Concrete Specimens

Days	Compressive Strength	Modulus of Elasticity	Modulus of Rupture	Splitting Tensile Strength
1	X	X	X	X
3	X			
7	X			
28	X	X	X	X

This test regimen allowed for full investigation of the mix designs. Since these mixes were intended for pretensioned concrete, the focus was on high early-age strength, namely one-day strength. The three-day and seven-day strength tests allowed for further refinement of the concrete strength gain curve. Finally, the 28-day test finished the test regimen. Since most concrete data are based on 28-day strengths, this seemed to be the most logical place to finish the testing.

3.1.4.1 Compressive Strengths

All compressive strength tests followed ASTM Test Procedure C39 [7]. The cylinders were moist-cured until the time of the test. The apparatus used in the test is shown in Figure 3.3.

Figure 3.3 Apparatus for Compressive Test

3.1.4.1.1 6000 psi Mixes

A graph of the age vs. strength curves for concrete mixes in the first iteration is presented in Figure 3.4. As can be seen, these strengths were highly variable. Particularly, Mixes 1-3 were totally undependable due to inexperience of the staff.

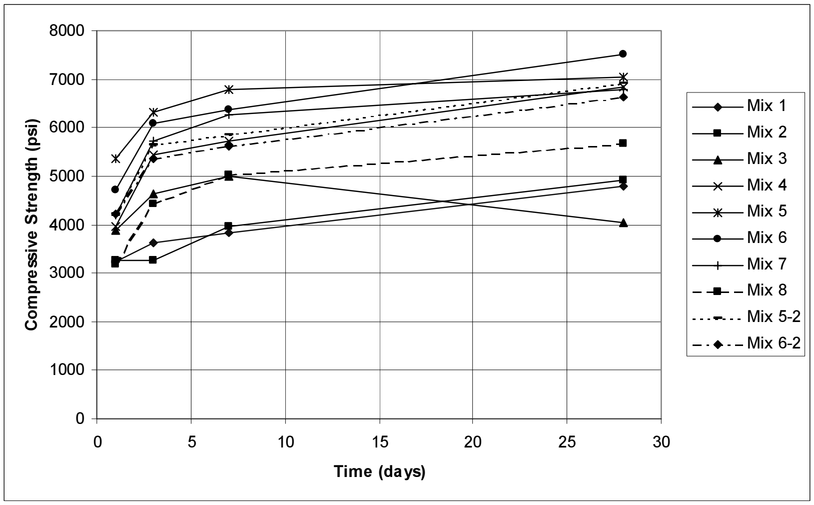

Figure 3.4 Compressive Strengths of Initial 6000 psi Mixes

The other mixes were better controlled and gave good results.

Most interestingly, all of the concrete except for Mixes 1 to 3 and Mix 8 reached the desired 6000 psi at 28 days. These results show that 6000 psi lightweight concrete can be easily attained.

Also, except for Mixes 1 and 2, the 1-day compressive strengths reached the 3500 psi goal to allow the precast plant to release the prestress.

However, these same mixes needed to exhibit repeatability to be considered candidates for use in the beams. Therefore, two mixes were chosen to be repeated. Figure 3.5 shows the results of two mixes which were repeated in comparison with their original results.

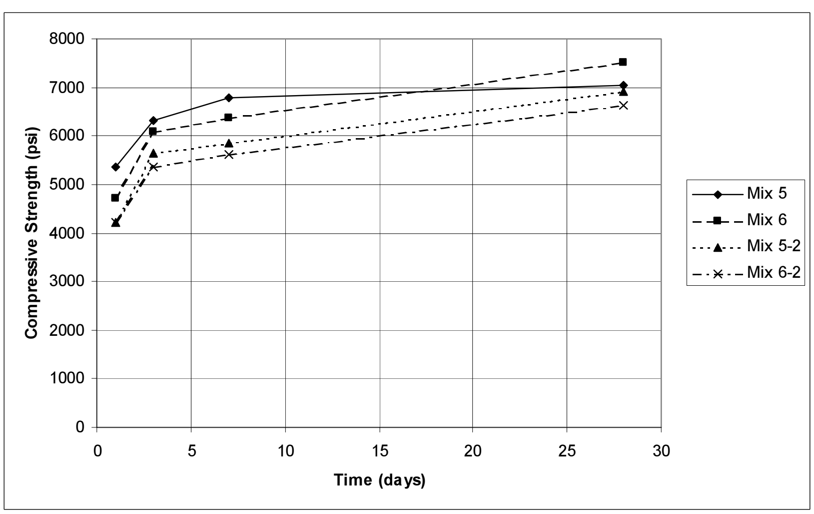

Figure 3.5 Compressive Strength of Repeated Mixes in Initial Series

Obviously, this figure raises problems. Neither of the repeated mixes reached the strength of the original ones. Also, the curve for Mix 5-2 did not match the slope of Mix 5. The slopes were nearly equal for Mixes 6 and 6-2.

There are a couple of possible explanations for this lack of repeatability. First and most likely, a difference in the moisture condition of the aggregate could have played a role in the strength. Due to the inexact nature of determining when the aggregate is in the SSD condition, the aggregate would often be added in varying surface conditions despite the best efforts of the staff. This affects the yield of the concrete. If the aggregate has different amounts of water absorbed but the same weight is placed in two mixes, a different volume of aggregate is placed in the two mixes, causing a different yield. This problem resurfaced later in the mixing process.

Second, the temperature and the ambient conditions could have caused a difference in the strength. On the warmer days when Mixes 5 and 6 were produced, the ambient air temperatures promote the reaction of the cement and the water, perhaps creating more strength.

3.1.4.1.2 8000 psi Mixes

The nominal 8000 psi mixes shown in Figure 3.6 gave disappointing results.

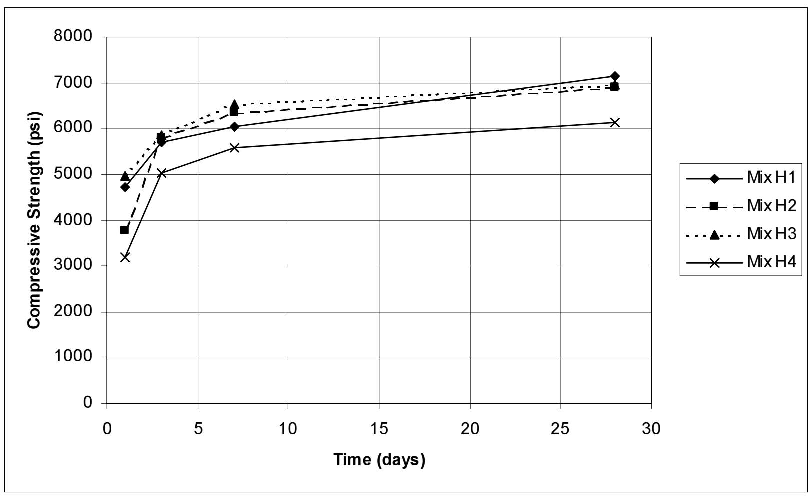

Figure 3.6 Compressive Strengths of 8000 psi Concrete Mixes

All the mixes were at least 1000 psi short of the target at 20 days. Since all of the concrete mixes used exactly the same amount of cement, the results were pretty consistent. Obviously, it was not enough cement to produce the required strength.

The fly ash did not play much of a role in strength formation. Mixes H2 and H4 both had fly ash replacement at 25% of the weight of cement. Comparing Mix H2 to Mix H1, fly ash barely reduced the compressive strength.

The aggregate proportion also did not play a large role in strength formation. Mixes H3 and H4 had much more coarse aggregate than did Mixes H1 and H2. The strength did not suffer at either end of the spectrum. Obviously, workability is going to play the largest role in determining the appropriate mix proportions.

3.1.4.2 Modulus of Elasticity

Another significant property was the elastic modulus. The modulus was determined using ASTM Test Procedure C469 [10]. The test setup is shown in Figure 3.7.

Figure 3.7 Test Setup for Modulus of Elasticity

The modulus was tested at 1 and 28 days. One day was chosen since this was when the concrete would be stressed due to pretensioning while the 28 day test would provide the elastic modulus used for service conditions.

Since bridge girders generally remain in the elastic range, this test holds a great deal of importance when calculating deflections and loss of prestress. Also, lightweight concrete generally has a reduced elastic modulus when compared to normalweight concrete. It was important to know the reduction in elastic modulus.

Figure 3.8 presents the modulus of elasticity results for the first batch of mixes.

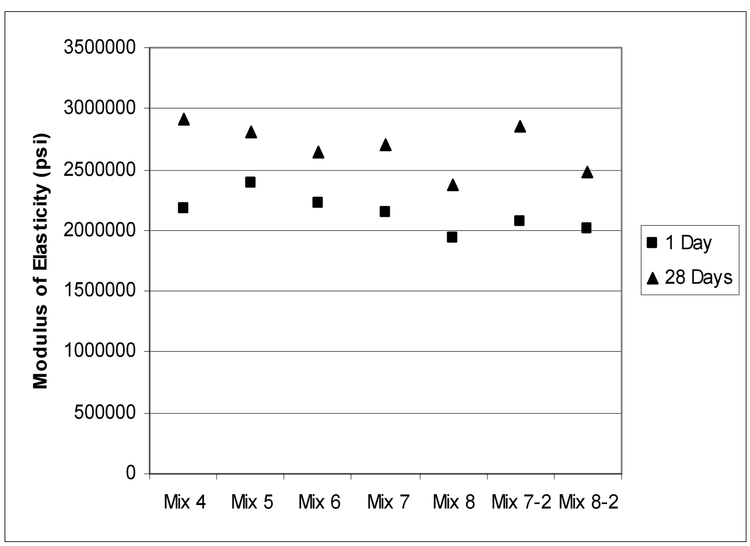

Figure 3.8 Modulus of Elasticity for 6000 psi Mixes in First Iteration

As can be seen from Figure 3.8 the modulus of elasticity generally was from 2,000,000 to 2,500,000 psi for 1-day age while the 28-day modulus of elasticity was from 2,500,000 to 3,000,000 psi.

The same results can be seen for the nominal 8000 psi concrete in Figure 3.9.

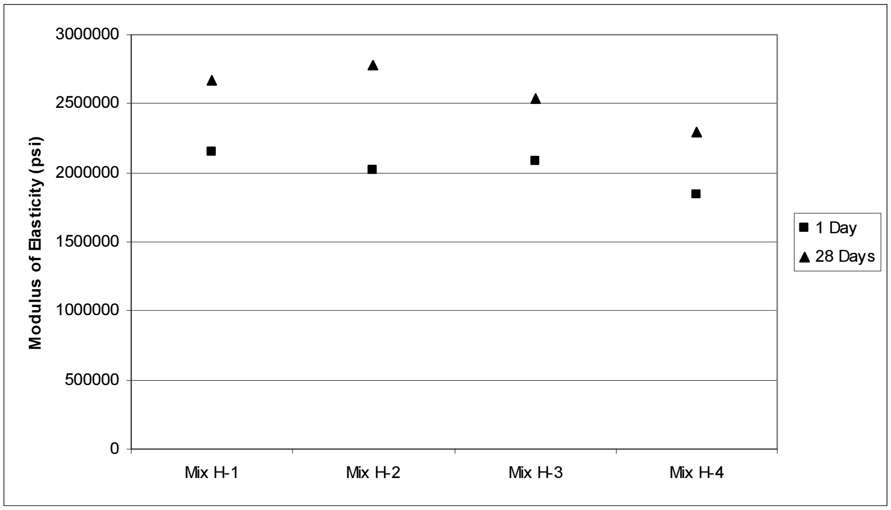

Figure 3.9 Modulus of Elasticity of 8000 psi Initial Concrete Mixes

These results were similar to the results for the 6000 psi concrete mixes. The moduli fell in the same ranges for both 1 day and 28-day tests. These results were predicted by Martinez [50] who showed that elastic modulus in lightweight concrete does not increase proportionally to strength gain in high-strength concrete.

3.1.4.3 Flexural and Tensile Properties

To complete the battery of tests carried out on the first sequence of concrete mixes, two different tensile tests were performed. Beams were fabricated for the Modulus of Rupture (MOR) test (ASTM test C78) [8] while cylinders were used for the split cylinder test (ASTM test C496) [11]. Both tests are very commonly used to measure tensile properties of concrete. Figure 3.10 and 3.11 show the test setups.

Figure 3.10 Test Setup for Splitting Tensile Test

Figure 3.11 Machine Used for Modulus of Rupture Tests

Since lightweight concrete was being used, much lower tensile values were expected. Since both concrete mixes being developed were for use in prestressed construction, tensile properties are very important in the determination of the allowable amount of prestress. Allowable amounts of prestress are controlled by AASHTO 9.15.2 [1]. Tensile strength of the concrete generally controls when the top fiber of the concrete at beam end goes into tension due to the eccentricity of prestress. Obviously, the tensile strength is very important.

The results for both MOR test and splitting tensile tests are presented in Figure 3.12.

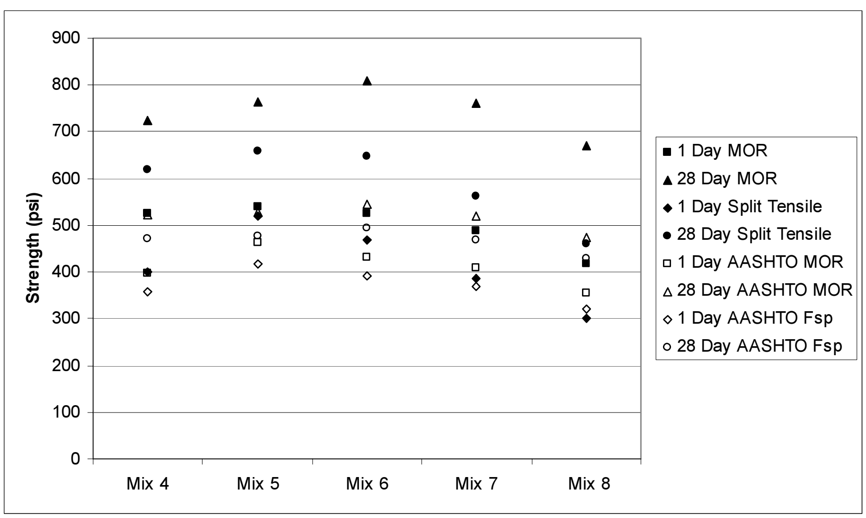

Figure 3.12 Tensile Tests for Initial 6000 psi Mixes

Both the MOR test and splitting tensile tests gained from 200 to 250 psi from one day to 28 days. Also, Mix 6 achieved the highest strength for 28 day MOR while Mix 5 had the highest splitting tensile strength at 28 days. Generally, as a rule of thumb, higher compressive strength meant higher tensile strengths.

Another interesting results can be seen when the results are compared to the AASHTO equations for MOR and splitting tensile strength. AASHTO 8.7.2 [1] allows the use of $6.3\sqrt{f_c^{'}}$ as an expression for MOR for sand-lightweight concrete. Figure 3.12 shows the comparison between the test results and the AASHTO predicted values for flexural strength. All five mixes outperformed the AASHTO equation predictions at both 1 day and 28 days.

The nominal 8000 psi mixes showed similar results. Figure 3.13 shows the tensile properties for the four mixes tested in this initial series.

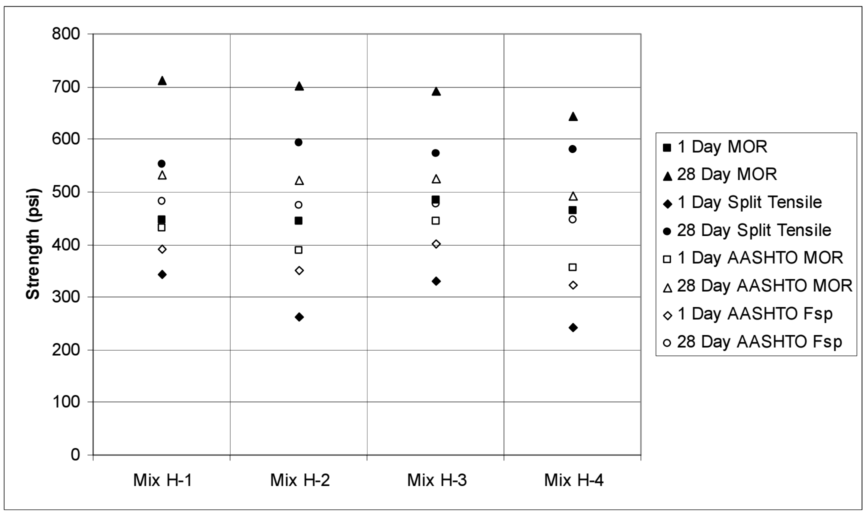

Figure 3.13 Tensile Properties for Initial 8000 psi Mixes

The results for the 8000 psi mixes were very similar to the results from the 6000 psi mixes. However, the results from the 8000 psi mixes did not reach as high a tensile strength as the 6000 psi mixes did. Also, the one day splitting tensile strengths were extremely low, lower than all of the results for the 6000 psi mixes. However, the modulus of rupture values met expectations. All values were satisfactory for the 28 day strengths. They also outperformed the AASHTO values for the modulus of rupture.

These results from the initial mixes of the concrete indicated that the tensile strength of the concrete should not be a main concern in the further iterations of the concrete mix design.

3.2 SECOND ITERATION

Once the initial iteration was complete, the approximate proportions for the 6000 psi mix were known. However, the mix proportions for the 8000 psi mix still needed to be found. Therefore, the main goal of the second iteration series was to refine the 6000 psi mix while increasing the cement content for the 8000 psi mix to reach the needed strength.

Five mixes were developed for test during this iteration. Although five mixes does not seem like a large number, these mixes were done three times apiece due to the addition of a new variable to the test program.

3.2.1 Modification of Variables

During this iteration, a new variable, coarse aggregate type, was added while the older variables were refined.

3.2.1.1 Water/Cement Ratio

From the results of the first iteration of mixes, it was recognized that the mixes were not reaching the 8000 psi requirement at 28 days. Since the water/cement ratio had been 0.32 for those mixes, the mixes the second iteration concentrated on lowering the water/cement ratio to achieve higher strengths. Table 3.3 presents the water/cement ratios used in this portion of the project.

Table 3.3 Water/Cement Ratios for Second Iteration

Mix Number	Water/ Cement Ratio	Pounds Cementitious Material/Cubic Yard
1	0.28	800
2	0.26	850
3	0.26	900
4	0.28	800
5	0.28	800

When compared to the values in Table 3.1 from the first iteration, it can be seen that these water/cement ratios were dropped significantly.

These lower water/cement ratios posed definite workability problems. The lowering of the water/cement ratios means that the use of superplasticizers and admixtures had to increase to counter the loss in workability.

3.2.1.2 Coarse Aggregate Factors

From the mixes of the first iteration, very little valid information about the best coarse aggregate fraction was received. Due to the handling of the coarse aggregate, there was very little uniformity in the moisture conditions of the aggregate. The results did not provide any indication as to the appropriate amount of coarse aggregate.

Therefore, for these five mixes, the amount of coarse aggregate and the proportion between the fine and the coarse was varied throughout the sequence of tests. Since these mixes would be closer to the actual proportions, it was hoped that the results from this sequence of tests would provide a good idea as to proportions needed.

Table 3.4 shows the coarse aggregate factors for these five mixes.

Table 3.4 Water/Cement Ratios and Coarse Aggregate/Fine Aggregate Proportions for Second Iteration Mixes

Mix Number	Water/Cement Ratio	Coarse Aggregate/ Fine Aggregate
1	0.28	1.1
2	0.26	1.25
3	0.26	1.1
4	0.28	1.25
5	0.28	1.15

Three separate coarse aggregate factors were chosen for these five mixes. The mixes had two different water/cement ratios. Each water/cement ratio had at least two different coarse aggregate factors that were tested along with it.

3.2.1.3 Types of Aggregate

One of the stated objectives of this project was to test at least three lightweight aggregates that are available for use in the Texas area. However, at the time of test, there apparently were only two aggregates that were widely available in Texas. These are Clodine and Streetman, both manufactured by Texas Industries (TxI).

In order to satisfy the requirement, a third aggregate needed to be obtained. With the help of TxI, a third aggregate (Western) was imported from Colorado. Shipments of three different aggregates were delivered to Ferguson Structural Engineering Laboratory and stockpiled outside. The three aggregates used were Clodine, Streetman, and Western.

Clodine is an expanded clay from the TxI plant south of Houston, Texas. It comes in a variety of maximum aggregate sizes ranging from 3/8 inch to ¾ inch. Figure 3.14 shows the appearance and maximum size of the aggregate.

The Clodine aggregate had a maximum size of 3/4 inch and was well-graded around the ¼-inch sieve. Figure 3.15 presents the grading curve for this aggregate.

The Streetman aggregate is an expanded shale produced south of Dallas at a TxI plant. Streetman is often used in precast plants as an aggregate for double-tee members for parking garage structures. It comes in a smaller size than does Clodine and the Western aggregate. Figure 3.16 shows the appearance and maximum size of the Streetman aggregate.

Figure 3.17 shows the grading curve. The maximum size is 3/8 inch, substantially smaller than the other two aggregates.

Figure 3.14 Appearance and Maximum Size of Clodine Aggregate

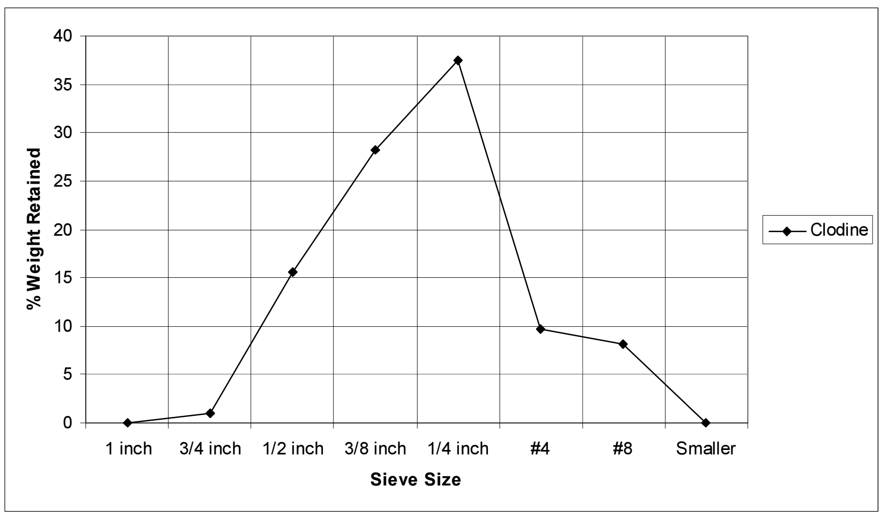

Figure 3.15 Grading Curve for Clodine Aggregate

Figure 3.16 Appearance and Maximum Size of Streetman Aggregate

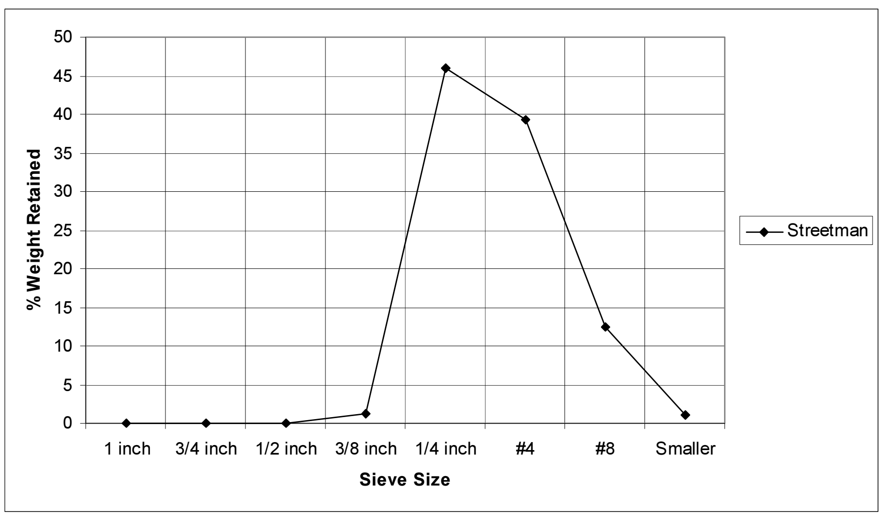

Figure 3.17 Grading Curve for Streetman Aggregate

The Western aggregate is an aggregate produced by a TxI subsidiary in Colorado. It also is an expanded clay like the Clodine. However, the shipment received was poorly graded, posing a problem as some grading is needed to produce a well consolidated mixture of concrete. Figure 3.18 shows the appearance and maximum size of the Western aggregate.

Figure 3.19 shows the grading curve for the Western aggregate.

Figure 3.18 Appearance and Maximum Size of Western Aggregate

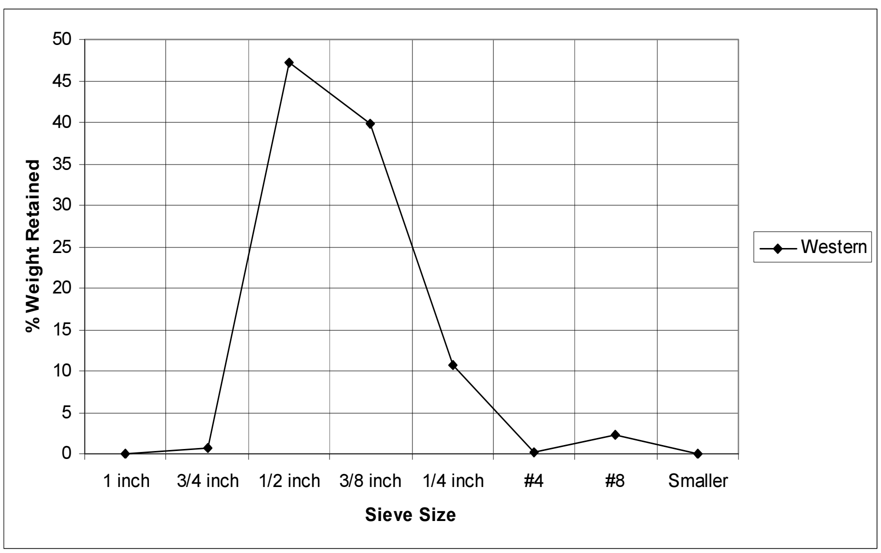

Figure 3.19 Grading Curve for Western Aggregate

The distribution of the Western aggregate was not very good. Almost all the aggregate was caught on the 1/2 inch and 3/8 inch sieves. The Clodine was much better graded.

3.2.2 Procedures

The same ASTM procedures were followed in the making of the concrete for this portion of the test program. However, there was one major change in the way the concrete was produced.

During the first iteration series of concrete production, the coarse aggregate was allowed to dry in the sun on a concrete deck. This left the aggregate exposed to both wind and sun, the two major environmental drying agents. This caused the aggregate to often be in widely varying moisture states when it was added.

Therefore, with the help of Don Reeves from TxI, a new system was developed in which the aggregate was dried to a consistent moisture state.

First, the aggregate was soaked in tubs for at least 24 hours. Then, the aggregate was placed in rectangular wooden beds with a screen bottom. The aggregate was also covered with black plastic sheeting to protect against wind and rain. Aggregate is shown drying in Figure 3.20. The aggregate was allowed to dry for an hour.

During this hour, the other concrete materials were batched. After the hour was up, a dry rodded unit weight (DRUW) was taken. The DRUW was checked against previous DRUWs taken during the test program. If the aggregate was within 2% of the previous DRUWs, it was allowed to proceed. If the aggregate was too wet, the aggregate was allowed to dry for an additional fifteen minutes and the DRUW was checked after this period of time. The aggregate never became too dry with this practice. Since DRUW is based on a constant volume, the coarse aggregate was at a consistent moisture state and was allowed to proceed.

Figure 3.20 Drying of Aggregate on Screened Bed

3.2.3 Results

The results were much more meaningful for this set of mixes due to the increased care shown in preparing the coarse aggregate and in mixing the concrete. The results overall were consistent and provided a good basis for further development of concrete.

3.2.3.1 Workability Issues

Since all three aggregate were expected to achieve approximately the same compressive strength, workability played a large role in differentiating the three aggregates. Workability was judged by how much superplasticizer was required to achieve the same slump as well as how well the concrete flowed after 30-45 minutes in the wheelbarrow.

Although quick placement is expected for mixes at the precast plant, the ability to remain reasonably workable for a 30 minute period played a large role in determining the best mix. The concrete needed to be placed in the forms, flow easily around the strands, and fill in the spaces without honeycombing. An overly stiff mix would not be able to do this.

To look at one aspect of workability, Table 3.5 presents the amount of superplasticizer added for every 100 pounds of cement.

Table 3.5 Average Superplasticizer Dosage Rates for Mixes with Three Aggregates

Aggregate	Dosage Rate (fl. oz./100 cwt.)
Western	4.70
Clodine	4.63
Streetman	5.06

Since all these mixes were dosed until they had the same amount of slump, approximately 9 inches, this table gives the average dosage rate needed to force the concrete mix to that point.

Table 3.5 shows that the mixes made with the Western and Clodine aggregates needed approximately the same dosage rate to reach the target slump while the Streetman needed about 6% more superplasticizer. The mixes with Western and Clodine aggregates were initially more workable before the addition of superplasticizer, which brought all the mixes to the same point of workability.

Since Streetman had a smaller maximum size than did the Western and Clodine aggregates, an equal weight of Streetman would have more surface area than a comparable weight of Western or Clodine. Therefore, the Streetman aggregate absorbs more mixing water in the mix, which causes the lower initial slump.

Also, another interesting observation was the fact that all three mixes had increased cohesiveness after thirty minutes. This observation is anecdotal as no test was performed to verify this. However, both project staff members noted that with all three aggregate after thirty minutes had expired, the concrete became gradually more difficult to place.

3.2.3.2 Yield Issues

Often, during the first iteration series of mixing concrete, the amount of concrete produced as measured by its unit weight did not agree with the theoretical amount. Obviously, a problem existed with the yield of the concrete mixes.

Discussions with Don Reeves of TxI confirmed that lightweight concrete has more variability with yield than does normalweight concrete.

Overyielding and underyielding are both major problems. Basically, if either one occurs, the concrete produced is not the one desired. Therefore, during this sequence of mixing, a test for fresh unit weight of the concrete was added to determine yield.

Expected yield for all mixes in this series was 3.5 cubic feet. Table 3.6 presents measured yield.

Table 3.6 Average Yields for Identical Mixes Produced with Three Aggregates

Aggregate	Average Yield (cf)	Variation (%)	Std. Deviation
Western	3.40	-3.31	0.12
Clodine	3.33	-4.74	0.05
Streetman	3.47	-1.93	0.10

These results indicated that inability to tightly control the volume of the concrete output is a major problem with lightweight concrete. This variation is due to the difficulty of controlling the state of water in the aggregates. In normalweight concrete, the coarse aggregate absorbs little water, causing the calculated yield to closely agree with results of volumetric measurements. In lightweight concrete, the aggregate can soak up or give off a large amount of water, which causes more problems and discrepancies in the volumetric yield prediction process.

The underyielding was caused by the variability in the volume of lightweight aggregate placed in the mix. Since the lightweight aggregate was batched by weight computed at SSD and since the aggregate was usually wetter than SSD, the volume of the aggregate actually placed in the mix was actually less than desired.

3.2.3.3 Mechanical Properties

3.2.3.3.1 Compressive Strength

Due to better control of the aggregate in this sequence of mixes, the results were much more meaningful.

Figure 3.21 shows the strength gain curves for the mixes with 0.28 water/cement ratio. Data for all mix designs are given in Appendix B. Expected compressive strength was around 6000 psi at 28 days. The major variation among the mixes was the amount of coarse aggregate used.

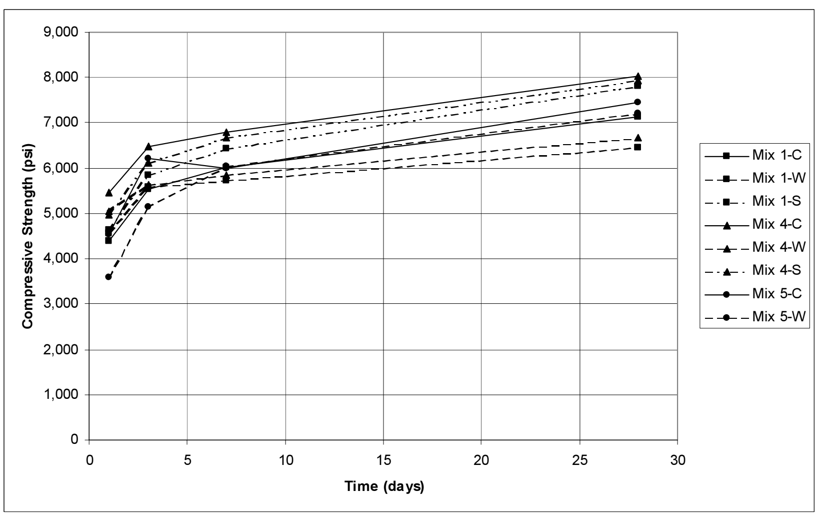

Figure 3.21 Age-Strength Curves for 0.28 Water/Cement Ratio Mixes from Second Iteration Mix Designs

The mixes produced with Western aggregates had much lower strengths at all ages than did the mixes produced with the other two aggregates. Also, the Western aggregate mixes did not gain as much strength from 7 to 28 days as did the other two aggregates.

Mix 4 seemed to perform the best of all the mixes. Both 4-C and 4-S produced the highest two strengths at all dates when strength was measured. Another notable fact was the strength that was eventually achieved. These two mixes almost reached 8000 psi at 28 days and were over 5000 psi at one day. They both easily satisfied the strength requirements for the 6000 psi mix.

The greater strength in this nominal 6000 psi mix seems to be related to the amount of coarse aggregate. In Mix 4, the coarse aggregate/fine aggregate proportion was the highest. This is not entirely logical since the coarse aggregate was lightweight aggregate that has a very low strength. High amounts of lightweight aggregate could pose workability problems.

Figure 3.22 shows the results from the 0.26 water/cement ratio mixes.

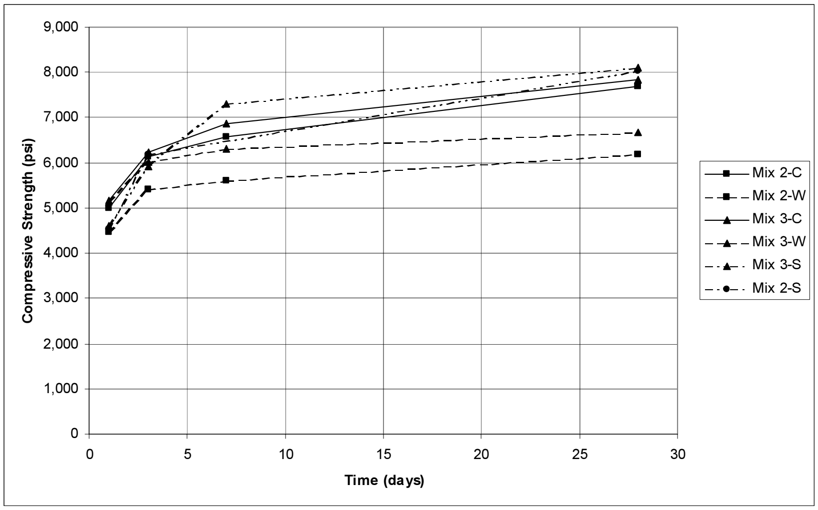

Figure 3.22 Age-Strength Curves for 0.26 Water/Cement Ratio Mixes from Second Iteration Mix Designs

Again, the mixes made with Western aggregates underperformed. They were significantly lower than the mixes made with Clodine and Streetman.

The strength development of the mixes with Clodine and Streetman were quite similar to Mix 1,4, and 5 from this iteration. The maximum strength varied from 7500 to 8000 psi while the 1 day strength varied were from 4500 to 5000 psi. These all satisfy the objectives quite easily.

From these results, it could be seen that the 6000 psi concrete would be easy to produce. The 8000 psi concrete seemed to be a tougher goal to reach. Many of the mixes hover right around the 8000 psi mark. However, they were produced under laboratory conditions, which cannot be expected at a prestressed plant.

3.2.3.3.2 Flexural and Tensile Properties

Once again, there were no specific goals for flexural and tensile performance for the mixes. However, they still were of interest due to the use of tensile properties in selecting prestressing force.

Figure 3.23 presents the MOR tests for this sequence of tests.

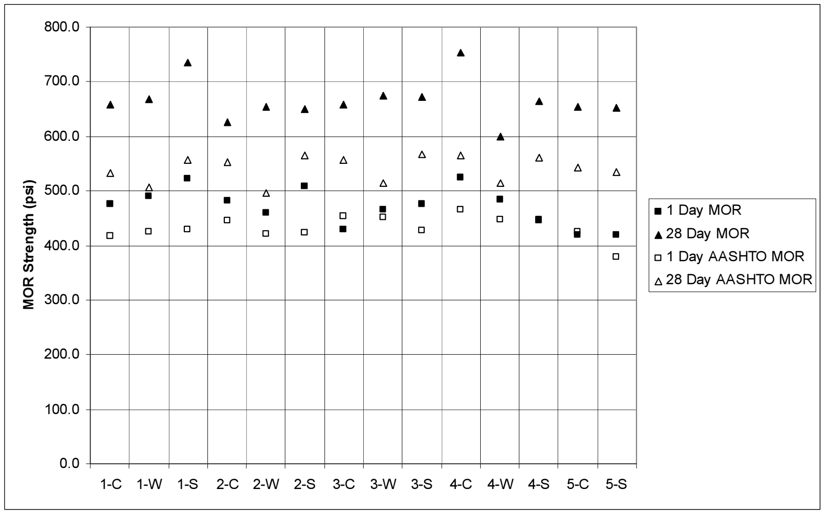

Figure 3.23 MOR Results for Mixes from Second Iteration of Mix Designs

The mixes all performed relatively equally. All MOR values ranged from 425 to 500 psi at 1 day and 650 to 700 psi at 28 days. Although no pattern could be discerned, the consistency was heartening since it indicated reasonable flexural strength could be expected from these mixes. Also, the mixes outperformed the AASHTO prediction for MOR at both 1 day and 28 days, showing that the mixes behaved well in flexure and that the AASHTO equation is conservative.

Figure 3.24 presents the splitting tensile results from these mixes.

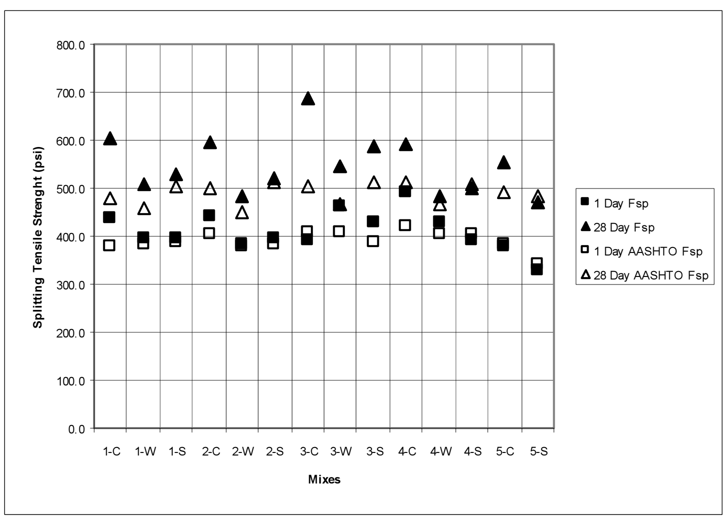

Figure 3.24 Splitting Tensile Results from Mixes Produced in Second Iteration of Mix Designs

Again, no strong patterns developed. There was no direct correlation between MOR and tensile strength. However, the concrete made with Clodine aggregate produced better results than the concrete made with the other two aggregates.

In modulus of rupture and compressive strength, the Streetman aggregate matched the Clodine aggregate in performance. Split tensile strength is the first indication that the Clodine aggregate performed better than the Streetman aggregate in any of the mechanical properties measured. Along with the results from the workability measures, the Clodine aggregate emerged from this sequence of tests as the favored aggregate for use in the concrete for prestressed applications.

Unit Weight Results

In the first iteration of concrete mixes produced, unit weight was not measured. However, after recognition of the problems with yield as well as the fact that the objectives called for development of a concrete with an equilibrium weight not more than 122 pounds per cubic foot (pcf), the unit weight of the concrete was measured at three different times, fresh, 28 days, and equilibrium.

Figure 3.25 presents the unit weight results.

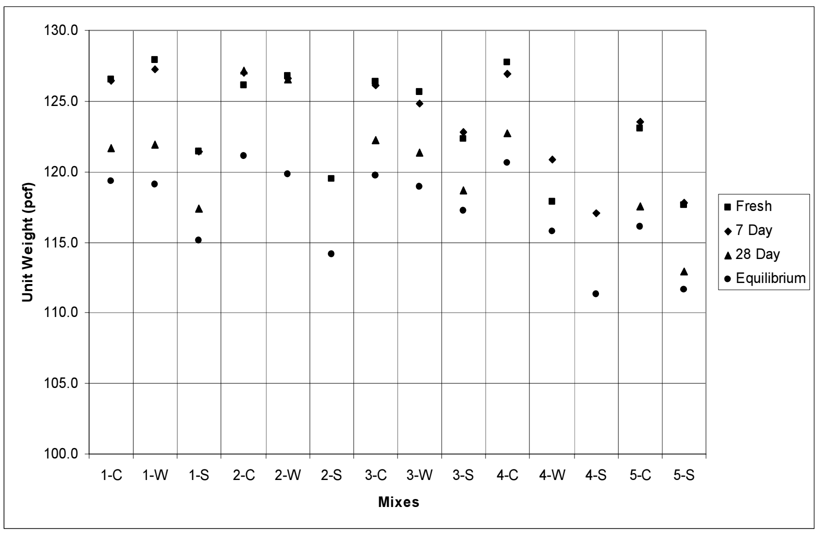

Figure 3.25 Weights of the Mixes in Second Iteration of Mix Designs

Most importantly, all of the mixes had equilibrium weights less than 122 pcf. The amount of weight these mixes lost from fresh to equilibrium conditions was considerable. This probably can be attributed to the amount of excess water in these mixes due to the moisture state of the aggregate. Since this water was not needed to hydrate the cement, it bled out. Despite the high initial weights, the 122 pcf goal was still satisfied. While all mixes underyielded, the mixes can be adjusted to shed enough water to reach the appropriate unit weight.

3.3 THIRD ITERATION

After completion of the first two iterations, the 6000 psi mix design was close to finalization while the 8000 psi mix design still had not been obtained. The first two iterations had narrowed down some decisions such as the aggregate that would be used for the project. Also, it provided excellent experience in producing concrete to ASTM standards. Therefore, on this third iteration, it was expected that the final mix design would be designed and produced.

3.3.1 Modification of Variables

Perhaps the biggest decision made after the second iteration involved choice of aggregate. The results overwhelmingly suggested that Clodine provided the best mix strength and workability. Due to its larger size than Streetman, the Clodine required less superplasticizer to be workable. However, its larger size did not take away its ability to gain strength. It was extremely competitive with the Streetman aggregate in producing the strength that was needed.

Also, the second iteration gave good indications about the amount of cement that would be needed to reach the 8000 psi goal. The 6000 psi goal had easily been reached with 800 pounds of cementitious material per cubic yard, but a dependable 8000 psi mix still needed to be achieved. It was obvious from the compression results that more cement would be needed and consequently, a lower water/cement ratio.

3.3.1.1 Amount of Cement and Water/Cement Ratio

From the previous two iterations, it was apparent that the water/cement ratio had been narrowed down to between 0.25 and 0.30 for both 6000 psi and 8000 psi mixes. Also, the 6000 psi goal had easily been reached, leaving the remaining decisions for the 6000 psi mix regarding how to find a workable mix for use in the precast environment.

However, the 8000 psi target was still posing problems. Although some of the mixes from the second iteration had exceeded 8000 psi, they had not exceeded it by enough to say with confidence that the mixes would be dependable 8000 psi mixes in production. Therefore, additional strength was still needed.

800 pounds or more of cement per cubic yard would probably be required to reach the 8000 psi goal.

Table 3.7 presents the amount of cement in the final iteration of mixes.

Table 3.7 Amount of Cementitious Material per Cubic Yard for Final Iteration Mixes

Mix Number	Lbs. Cementitious Material/Cubic Yard	Water/Cement Ratio
Mix F-1	550	0.36
Mix F-2	600	0.35
Mix F-3	600	0.35
Mix F-4	600	0.35
Mix F-5	657	0.33
Mix F-6	800	0.28
Mix F-7	978	0.25
Mix F-8	978	0.25
Mix F-9	978	0.25

Mixes F-1 through F-5 focused on finalizing the 6000 psi mix. Their weights of cement were similar to the mixes in the first and second iterations shown in Tables 3.1 and 3.4. Mix F-1 explored the possibility of using less cement, thereby making the concrete more economical. Mixes F-2 through F-4 were refinements of earlier mixes. Mix F-5 was a response to underyielding problems.

Mixes F-6 through F-9 were aimed at solving the 8000 psi strength problem. Mixes F-7 through F-9 were based upon a mix used in an earlier high-strength normalweight concrete project by John Myers [57]. The volume of normalweight aggregate was replaced by an equal volume of lightweight aggregate.

3.3.1.2 Chemical Admixtures

For this sequence of the project, Daratard-17 and ADVA Superflow were again the two chemical admixtures used. However, it was expected that larger amounts of superplasticizer and retardant would be required by the large amount of cement being used in Mixes F-7 through F-9. From 7 to 9 inches of slump was still the guide for the amount of superplasticizer.

3.3.2 Procedures

The same mixing procedures as used in the second iteration were followed. A rodded unit weight of the aggregate was taken to check that the aggregates were in a consistent moisture state when placed. There was a little more consistency for this portion of the project since Clodine was used as the aggregate for all of the mixes. Thus, the variability inherent to the aggregate was reduced to a manageable level.

The aggregate was dried the same way, by placing it on a screen and allowing it to dry with the aid of gravity. The other materials were all batched in the same way.

The only significant difference came in the type of cylinders used. Previously, 4 inch x 8 inch cylinders had been used throughout the project. They minimized concrete usage as well as made placement easier. However, along with the 4 x 8 cylinders, 6 inch x 12 inch cylinders were also cast. They would provide an important check for the tests since the larger cylinders are usually the method of field control and generally produce lower strengths.

3.3.3 Properties

3.3.3.1 Workability

Mixes F-1 through F-6 designed for 6000 psi did not pose any workability problems. They performed similarly to the mixes made with Clodine from the second iteration, requiring approximately the same amount of chemical admixture for the desired workability. Also, they remained workable for the same period of time, about thirty minutes.

On the other hand, Mixes F-7 through F-9 with 978 pounds of cement per cubic yard were another matter. As expected, they required more superplasticizer due to the increased amount of cement. However, this did not mean that they also needed an increased dosage rate. Table 3.8 shows the dosage rate for each of the mixes in this sequence.

Table 3.8 Dosage Rates of Superplasticizer for Third Iteration

Mix Number	Dosage Rate (fl. oz.)/100 lbs. Cement
Mix F-1	13.1
Mix F-2	5.4
Mix F-3	5.4
Mix F-4	7.2
Mix F-5	4.9
Mix F-6	5.4
Mix F-7	5.5
Mix F-8	5.5
Mix F-9	5.5

The dosage rate overall stayed fairly constant for these mixes. Except for a few aberrations (Mix F-1 and Mix F-4) which can be blamed on experimental error, the dosage rate generally ran about 5.5 fluid ounces per 100 pounds of cement.

However, the required amount of retardant changed for the mixes with higher amounts of cement. On the first of these mixes, Mix F-7, the concrete set very quickly. The concrete also appeared extremely sticky and was difficult to scoop out and place in cylinders after a short time. Again, although precast plants generally place their concrete very quickly, this could pose a problem since more than 15 minutes of dependable workable time is needed.

To combat this problem, retardant dosage was increased. Beforehand, a negligible amount of retardant was added to the mix. However, after encountering this problem in Mix F-7, the retarder dosage was increased for Mix F-8 from 1.1 fluid ounces per 100 pounds cement to 2.75 fluid ounces per 100 pounds cement. In Mix F-8, this seemed to adequately restrain the reaction of the cement with the water enough to place all the concrete. Retardant thus became an integral part of the chemical admixture mix for the 8000 psi goal.

3.3.3.2 Mechanical Properties

3.3.3.2.1 Compressive Strength

For these results, only data from the 6 inch X 12 inch cylinders will be presented as these data are more reliable and accepted. Figure 3.26 presents the compressive strengths for the mixes designed for 6000 psi.

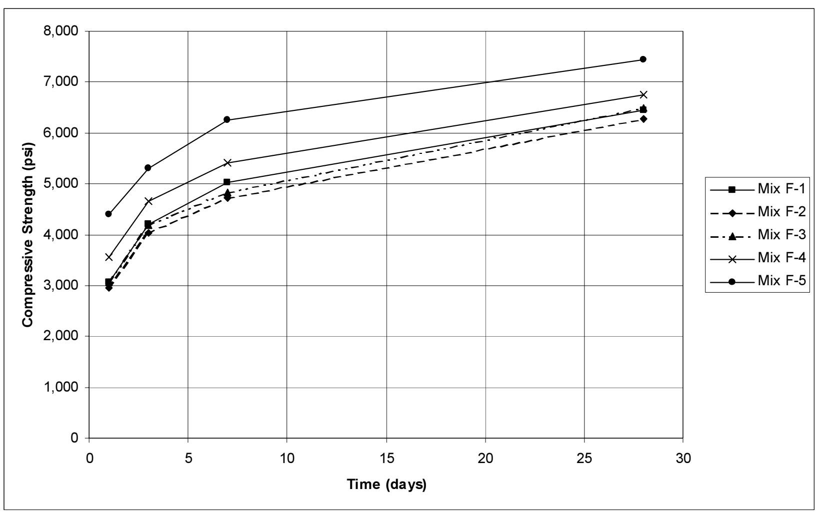

Figure 3.26 Age-Strength Curves for 6000 psi Mixes in Third Iteration

Although the results were down somewhat from those of the second iteration, a couple of mixes still were performing well above specifications. Mixes F-4 and F-5 would both be satisfactory for use as the 6000 psi mix in the field. However, due to its increased one-day strength, Mix F-5 is recommended since it provides the best reserve for early precast yard release.

Of more interest was the performance of the 8000 psi mixes. The hope was that the increased cement content would help reach the 8000 psi goal.

From Figure 3.27, it is seen that the three concrete mixes with 978 pounds of cementitious materials per cubic yard reached 8000 psi. Mix F-6, a revision of an 800 pounds of cementitious materials mix, almost reached 8000 psi. Mixes F-7 and F-8, which were exactly the same, except that different amounts of retardant were used to control the workability, performed nearly the same. Mix F-8 lagged a bit due to the extra retardant but eventually caught up at 28 days. Both mixes were clearly above the 8000 psi level in the laboratory but with somewhat marginal reserve for plant conditions. Mix F-9 had an increased amount of coarse aggregate but was marginal even under laboratory conditions. Therefore, Mix F-8 was chosen as the 8000 psi mix for use in the precast yard for the production of the beams.

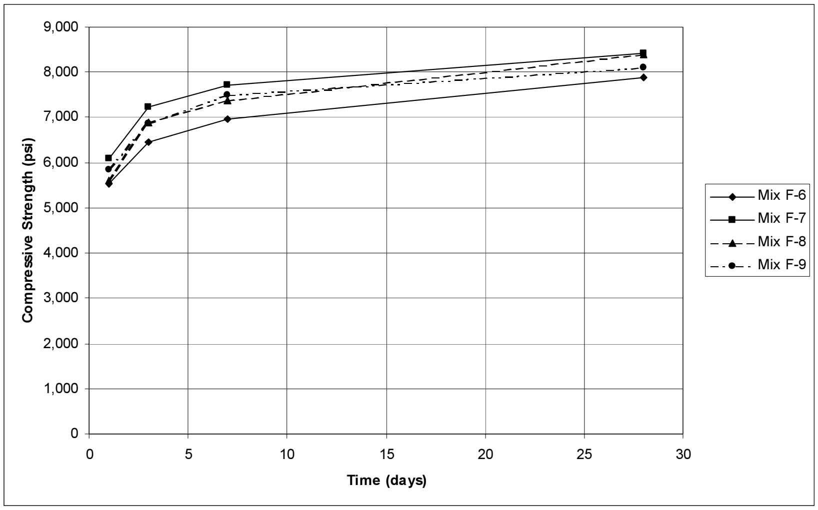

Figure 3.27 Age-Strength Curves for 8000 psi Mixes for Third Iteration

3.3.3.2.2 Tensile Properties

The MOR tests were not done for these specific mixes. The data from the first two iterations gave a good idea of the values for lightweight concrete. Therefore, the splitting tensile test was the only one performed on these specimens. This test was still important since it plays an important role in the design of the tensile reinforcement of beams.

The results from these tests as shown in Figure 3.28 were different from earlier experience. This maybe can be attributed to the use of 6 inch x 12 inch cylinders. Also, a refinement in the testing procedure of slowing down the loading rate to ASTM standards also played a large role.

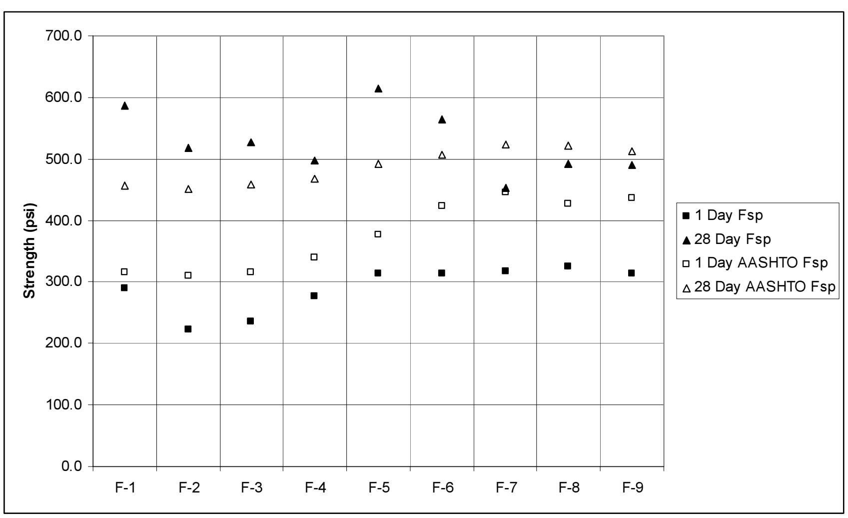

Figure 3.28 Splitting Tensile Strengths of Mixes in Third Iteration

The splitting tensile strengths were lower than those expected based on results from the second iteration. The 1 day strengths with the 6 X 12 inch cylinders were usually around 300 psi although the second iteration with 4 X 8 inch cylinders generally produced strengths around 400 psi. Also, the 28-day strengths came out to be around 500 psi, which was a little lower than the usual 550-600 psi of the second iteration. These results can be attributed to the change in cylinders size and to the slowing of the loading rate used in this round of tests.

Also, the concrete was allowed to dry too much before the test. Martinez [50] showed that splitting tensile strengths of dry cylinders are only 50-60% those of wet cylinders. These cylinders had dried for 45 minutes prior to testing, possibly affecting the strength.

3.3.3.2.3 Unit Weight Results

Again, the unit weights of the concrete mixes were taken at various times to make sure they met the goal of being not more than 122 pounds per cubic foot (pcf) at equilibrium weight. Figure 3.29 presents these results.

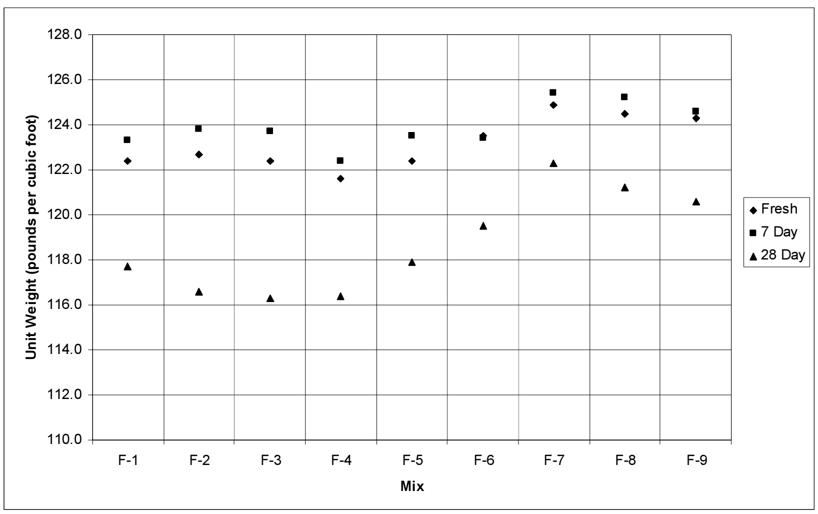

Figure 3.29 Unit Weights of Mixes from Third Iteration

Although equilibrium weight was not measured, the unit weights shown in Figure 3.30 indicate that all the 28-day weights meet the 122 pcf requirement. From prior experience in the first two iterations, it is known that the concrete loses weight from 28-day weight to equilibrium. Therefore, these mixes performed well.

The mixes using more cement (F-7 through F-9) had larger unit weights due to the reduction of lightweight aggregate volume. However, they still met the requirement.

CHAPTER 4: 6000 PSI MIX FIELD TRIALS

4.1 MIX PROPORTIONS

Mix F-5 was chosen for the 6000 psi mix for the precast plant field trials. It combined dependable one-day strength with acceptable workability properties. Performance during mix trials indicated this mix could be repeated with high confidence. Table 4.1 presents the theoretical mix proportions for the 6000 psi mix for one cubic yard of concrete.

Table 4.1 Mix Proportions per Cubic Yard of 6000 psi Mix

Component	Proportion
Cement	504 lb
Fly Ash	168 lb
Lightweight Aggregate	1264 lb
Sand	1149 lb
Water	222 lb
Daratard-17	12 oz
ADVA Superflow	34 oz

Mix F-5 was scaled up from Mix F-3, which had yielding problems. Since Mix F-3 underyielded, the mix proportions were increased by the percentage that Mix F-3 tended to underyield.

The mix is a 7.15 sack mix when including the fly ash in the cementitious materials. The fly ash comprises 25% of the cementitious material by weight. The water/cement ratio is 0.33. This mix assumes that the lightweight aggregate will be added in the saturated surface dry condition after submersion or sprinkling for at least 24 hours. The proportions of the chemical admixtures are adjustable to optimize their use. The amounts reported here provide a guideline for approximate amounts to be used. However, more or less superplasticizer might be required depending on weather conditions.

4.2 PROPERTIES

4.2.1 Workability

Among the biggest concerns at the initiation of this project was achieving adequate workability. Due to the high cement contents, some difficulty was expected. However, in lab mixing, this mix performed admirably in workability measures. The mix provided enough time for placement in forms at a prestressed plant as long as mechanical vibration was available, a standard practice at precast plants.

4.2.1.1 Slump

Due to the demands of placement, this concrete needed to flow. Since precast concrete plants often use mechanized carts with hoppers and chutes (sidewinders) for placement, high slump concrete was needed to aid in removal from the sidewinder into the forms. A sidewinder is shown in Figure 4.1.

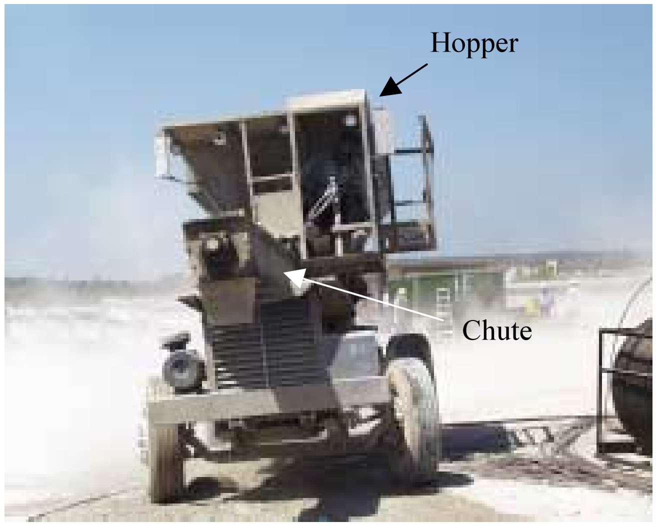

Figure 4.1 Sidewinder at Heldenfels Precast Plant

The sidewinders do not have any way to mix the concrete or vibrate it. The concrete must easily slide out of the sidewinder into the forms. Because of this the slump for this concrete was specified between 7 and 9 inches. A concrete with this amount of slump will be akin to a thick milkshake. This slump was achieved through the addition of an appropriate amount of superplasticizer. Before the superplasticizer is added to the mixer, this concrete mix had approximately ½ to 1 inch of slump.

In the trial mixing period, the 6000 psi mix had 6 fluid ounces of ADVA Superflow added to achieve 6.5 inches of slump. This amount was appropriate for five cubic feet of concrete. Adjusted for a cubic yard of concrete would give 34 fluid ounces of superplasticizer. This was the amount used in the final mix. However, this amount is not a constant. Due to the inherent variability of concrete with climatic conditions, some change in amount of superplasticizer should be expected

4.2.1.2 Finishability

The other aspect of workability lies in the finishability of the concrete. This is a measure of how well the concrete fills edges and corners and how well the concrete takes a smooth surface.

For the application in pretensioned girders, the concrete did not need to finish smoothly. Actually, a rough surface is preferable since that surface promotes good bond between the deck and girder, allowing good transfer of the horizontal shear between the deck and girder. No objective measure exists for finishability. Relying on first-hand observation is the only way to have any idea as to how well the concrete finishes.

During the trial mixes in the laboratory, finishing the concrete to a smooth surface was difficult. Even with extreme caution and care, some aggregate was still visible in the top portion of concrete surfaces after finishing of the concrete. The aggregate was completely covered by paste and only protruded above the surface by approximately 1/16 inch, a relatively minor amount. This finish was acceptable for the girders due to the rough surface desired for good bond. It might not be acceptable for a finished deck surface. The concrete could be pushed and shaped into position by a trowel. Also, fine troweling of the surface resulted in the rough surface described.

4.2.1.3 Consistency

Another important characteristic of the concrete was its consistency. This property is closely related to slump and finishability, yet encompasses some different aspects. After completion of rotation in the laboratory mixer, the concrete retained a slump of 7 to 9 inches for fifteen minutes. There was no segregation of the paste from the aggregate and the aggregate remained properly coated for the whole time.

After thirty minutes, the concrete started to bind. The concrete was very cohesive and difficult to scoop from the wheelbarrow. The concrete could still be vibrated into place. However, the difficulty in handling had increased. Mixing with a shovel after this time alleviated the problem somewhat. The concrete remained in this state for a substantial period of time, approximately thirty minutes. Since precast plants place their concrete extremely fast, this provided an acceptable window of time for the concrete.

4.2.2 Mechanical Properties

Along with the workability properties, the mechanical properties of the concrete played a large role in its acceptance for usage. Mix F-5 satisfied the objectives placed upon it at the beginning of the project.

4.2.2.1 Compressive Strength

Obviously, the most important aspect to be satisfied was the ultimate compressive strength of the mix. At the same time, the mix also needed to have an adequate strength at one day for release in the precast plants. Figure 4.2 presents the age-strength relationship for the 6000 psi mix.

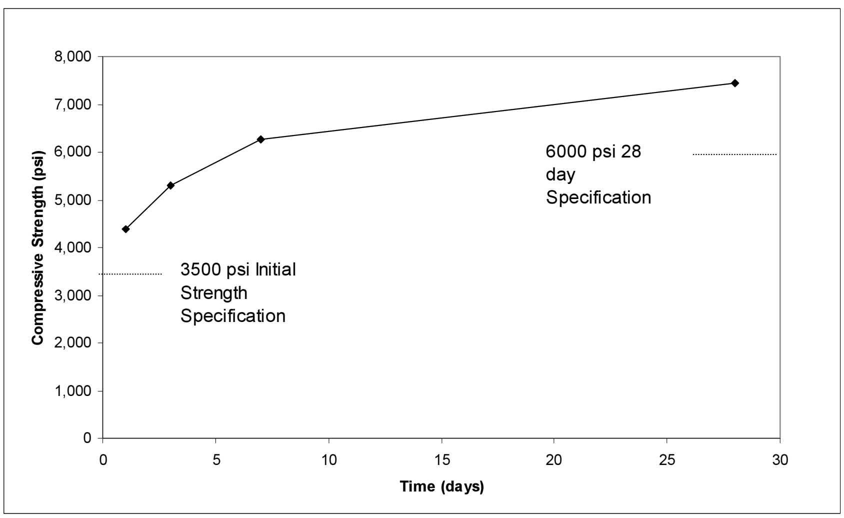

Figure 4.2 Age-Strength Compressive Strength Relationship for 6000 psi Mix

This mix easily satisfied the first requirement, namely that the mix had to achieve at least 6000 psi at 28 days. In fact, it performed much better than that, reaching 7400 psi. This was largely due to the companion requirement that the one-day strength needed to be at least 3500 psi for initial release of the strands. The chosen mix had to provide a good margin for the release strength.

There were other interesting aspects to the performance of this mix. The concrete continued to gain substantially in strength after seven days, a somewhat surprising result considering the use of Type III cement. In fact, the strength gain is quite gradual and consistent, indicating that this concrete probably gained more strength past 28 days.

This gain in strength can be attributed to the use of fly ash. The use of fly ash contributes to the long term strength gain of the concrete since it is similar to cement in its chemistry but reacts at a much slower rate. Thus, the fly ash helps increase the ultimate strength of the concrete at later ages.

4.2.2.2 Modulus of Elasticity

Modulus of elasticity plays a large role in determining the deflections of the member. Overall, lightweight concrete has values of modulus of elasticity less than those of normalweight concrete. However, of more interest is comparing the values to other tests done on lightweight concrete. In Figure 4.3, the data from the 6000 psi mix is compared to data from four other studies. All involved lightweight concrete and focused mainly on high-strength concrete.

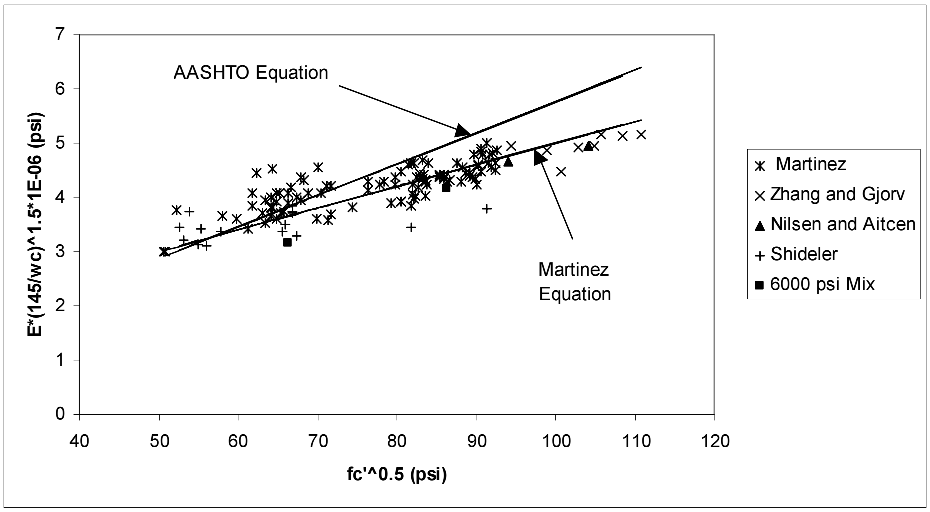

Figure 4.3 Comparison of Moduli of Elasticity days for 6000 psi Mix

Figure 4.3 shows how well the concrete chosen for the field trials compared to concretes used in other projects. This graph substantiates the assertion of Martinez [50] that the AASHTO equation for the modulus of elasticity is not accurate for high-strength lightweight concrete. It also shows that the equation developed by Martinez is applicable.

The selected 6000 psi concrete performed well within the scatter band of the Martinez [50], Shideler [70], and Zhang and Gjørv [80] data which were similar in strength and composition to the concrete developed in this project. Of the two data points for the 6000 psi concrete of this study, the more important data point, that for the concrete at 28 days, lies close to the Martinez Equation 2.1.

4.2.2.3 Tensile Strength

For Mix F-5, the splitting tensile strength was determined using 6 X 12 inch cylinders. In Figure 4.4, the splitting tensile strength of the 6000 psi mix is compared to data from Martinez [50] and Zhang [80]. Both Martinez and Zhang used all-lightweight concrete which have lower values of splitting tensile strength. Therefore, both AASHTO equations for all-lightweight and sand-lightweight concrete are included.

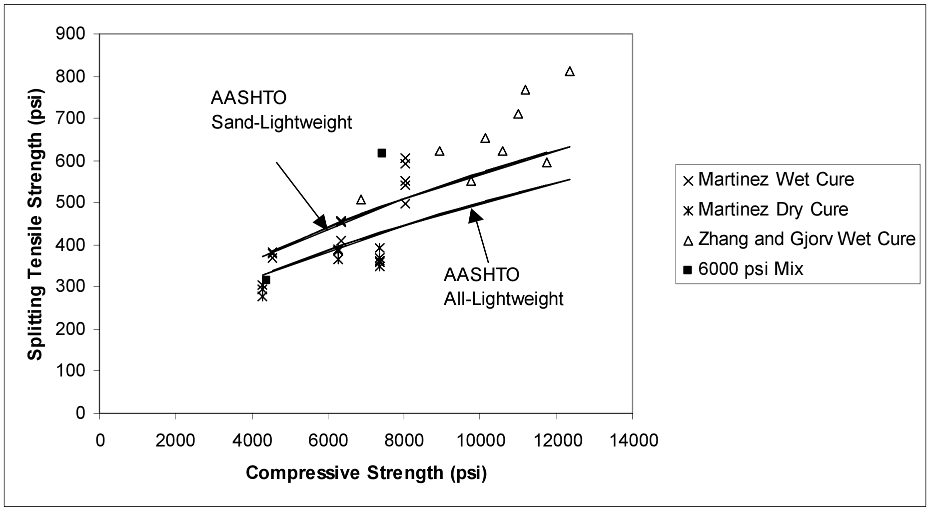

Figure 4.4 Comparison of Splitting Tensile Values for 6000 psi Concrete

Both the 1-day and 28-day values are included. The 28-day value is of more interest. The 6000 psi concrete performed well. The splitting tensile strength was well over the AASHTO predicted value at 28 days. This was expected since the AASHTO equation is generally conservative for splitting tensile strength. From this result, it was decided that the 6000 psi concrete mix has adequate tensile strength.

4.2.3 Creep and Shrinkage

4.2.3.1 Creep

One of the biggest concerns with lightweight concrete is its creep under sustained load. Since this lightweight concrete will be placed under sustained load from the prestress placed into the beam, this is a very important property. The creep of the 6000 psi mix was tested according to ASTM C512-87 [12]. Figure 4.5 shows loaded creep specimens.

Figure 4.5 Creep Cylinders

The strains were measured with a DEMEC device. Metal disks are placed approximately 8 inches apart. Then, a gauge is used to measure changes in this distance. For these cylinders, three different measurements were taken on opposite sides of the cylinder and averaged to find the creep.

In this report, the results from the cylinders whose initial loading was at 2 days and 7 days are presented. The cylinders were loaded to 40% of their ultimate strength at that age as per ASTM C512-87 [12]. These cylinders are of more interest due to the fact that most pretensioning is introduced into the concrete at early ages, 1 or 2 days. Also, this report includes the creep plus shrinkage data for these concrete mixes. No room was available with the appropriate ASTM conditions for measuring creep only. Therefore, the specimens had to be placed in ambient air conditions. They were protected from the elements, but not from changes in temperature and humidity. Therefore, these data include drying shrinkage. The creep and shrinkage behavior could not be separated since two different size specimens were used.

Figure 4.6 presents the creep data from the 6000 psi concrete mix.

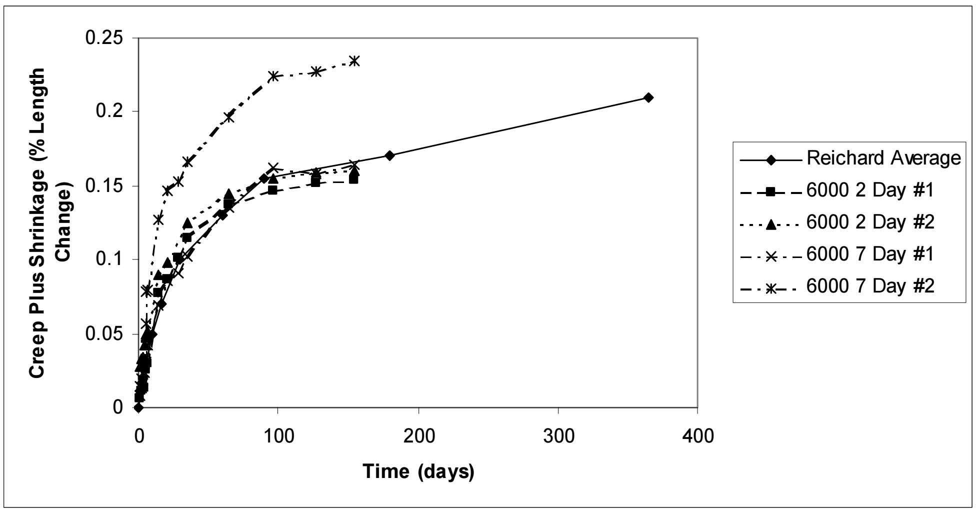

Figure 4.6 Early Age Creep Plus Shrinkage of Cylinders

The creep plus shrinkage is consistent with the Reichard [64] average prediction of lightweight concrete creep plus shrinkage behavior. Three of the four cylinders analyzed fall nearly on top of the Reichard average prediction curve. The 6000 7 day #2 cylinder is considered an outlier and should not be considered when making judgments about the behavior of this mix. Figure 4.6 shows that creep of the 6000 psi mix is normal for lightweight concrete and should not be a factor that causes concern.

Another measure of creep is the creep coefficient. The creep coefficient is defined as the following [58]:

$$C_u = \frac{\varepsilon_{cu}}{\varepsilon_{ci}}$$ **Equation 4.1**

where ε_{ci} is the initial elastic strain and ε_{cu} is the additional strain resulting from creep.

Table 4.2 presents the other measures of creep.

Table 4.2 Five Month Creep plus Shrinkage Performance of 6000 psi Mix

Age at Loading	Initial Elastic Strain (microstrain)	Creep Coefficient
2 days	702.5	3.19
2 days	516.4	4.09
7 days	827.1	2.96
7 days	1084.5	3.16

Table 4.2 shows a different story.

Here, 6000 psi 2 day cylinder #2 is the outlier, different from when the data was compared to Reichard [64]. However, it should be noted that for this cylinder there was a great deal of strain recorded in the first 2 hours, suggesting that there was a misreading of the immediate elastic strain. Therefore, this cylinder cannot be considered a true picture of the creep plus shrinkage.

Table 4.2 suggests that the creep coefficient after five months is approximately 3.1. This number falls on the high end of the normalweight concrete scale, which generally ranges from 1.6 to 3.2 [58]. However, as seen when compared to data from Reichard [64], the creep plus shrinkage of this concrete was normal.

4.2.3.2 Shrinkage

Shrinkage can be another important problem with lightweight aggregate concrete. Figure 4.7 shows the shrinkage of both dry and wet cured concrete over a five month period.

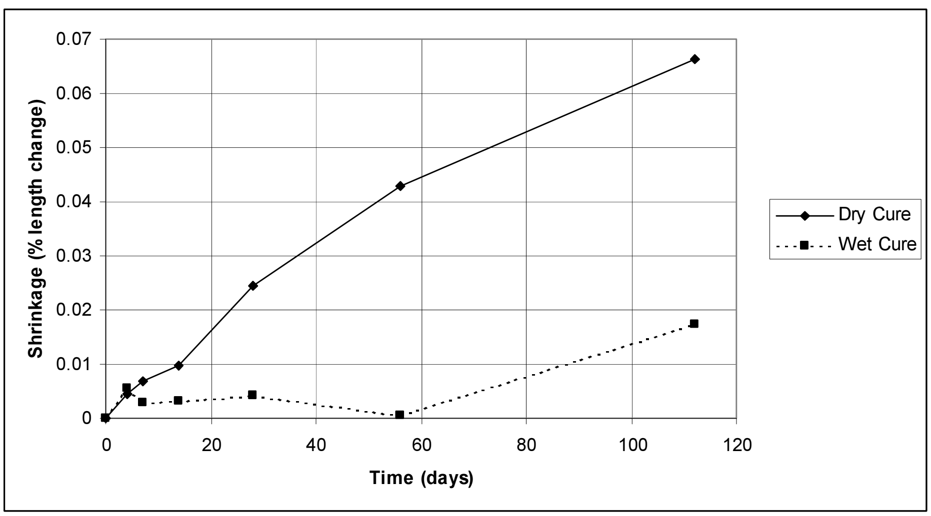

Figure 4.7 Shrinkage of 6000 psi Mix

When compared to Reichard [64], these shrinkage values were average for lightweight concrete. Reichard tested 24 different lightweight aggregates in concrete for creep and shrinkage. He found that lightweight aggregate concrete that was dry cured at a constant temperature and humidity had shrinkages from 0.02% to 0.08% at 90 days. From the 6000 psi data, the shrinkage at 90 days in constant temperature and humidity was approximately 0.055%. Also, Reichard showed that lightweight concrete shrinkage generally plateaued at approximately 150 days.

Thus, the shrinkage of the 6000 psi mix was average. The results agreed with data from Reichard [64], upon which ACI Committee 213 [2] bases its creep and shrinkage recommendations.

4.3 JOBSITE PERFORMANCE

One of the most important aspects of this project was to determine how well the concrete performed when mixed and placed at the precast plant. The actual use of the concrete would gauge the performance of the concrete and help determine the ability of precast plants to handle the use of lightweight concrete for prestressing.

At the plant, the aggregate was sprinkled for 48 hours prior to initial use. For the 6000 psi mix, two different trials were run. On the first day, two different 20 foot beams designed to represent the highest level of reinforcement congestion likely to be encountered in beams in practice were produced. One used this 6000 psi concrete mix. The other used a companion 8000 psi mix. This allowed determination of the approximate behavior of the concrete before placement in full length specimens.

After that, two 40 foot beams were produced using the 6000 psi mix. On the same day, a normalweight 40 foot beam was also cast.

4.3.1 Workability

At the plant, the slump of the concrete was again controlled through the use of superplasticizer to achieve the desired slump. For the 20 foot beam, one three cubic yard batch of concrete was used to produce the concrete. For this batch, a dosage rate of 5.96 oz./100 pounds of cementitious material (cement plus fly ash) was used. This dosage rate was a little larger than predicted from laboratory calculations, but this was expected. The concrete produced a slump of 6.5 inches, a little lower than wanted. However, the concrete performed extremely well during placement in the highly congested beams as it required little vibration to be placed. Overall, the concrete for this beam performed well and verified the concrete mix design process of the first part of this project.

After observing the initial trial 20 foot beam, it was felt that the mix was ready for placement into the two 40-ft beams that would actually be load tested later in the project. To produce these beams, two three-yard batches of concrete would be mixed. The proportions were expected to be the same as for the 20 foot beam.

The first batch again used 150 ounces of superplasticizer, a dosage rate of 5.96 oz./100 pounds of cement. The slump for this batch when it left the batch plant was 9 inches. However, after transport, this concrete had lost 1 inch of slump and was down to 8 inches.

The slump of the first batch is shown in Figure 4.8.

Figure 4.8 Slump of First Batch of 6000 psi Concrete

This concrete proved to be slightly thin (not cohesive). To correct this problem, the next batch of concrete was given 120 ounces of superplasticizer. This dosage solved the problem as the concrete retained its slump of 7 inches on the way to the prestressed bed. Figure 4.9 shows the slump of the second batch.

Figure 4.9 Slump of Second Batch of 6000 psi Concrete

As was expected, the beams produced from the 6000 psi mix concrete exhibited an excellent finish. No problems were seen with honeycombing or voids in the concrete. The finish was comparable to other normalweight girders produced at the Heldenfels precast plant. Figure 4.10 shows the finish of the 6000 psi beams.

Figure 4.10 Finish of Girder Made with 6000 psi Mix Concrete

Overall, the workability of the concrete proved to be excellent. The workers reported no problems with placement as the concrete finished well in the bed. Also, the workers did not have to do a great deal of work to get the concrete placed into the forms. This particular mix proved to be excellent for prestressed applications.

4.3.2 Compressive Strength

Obviously, determination of the mechanical properties of this concrete plays the greatest role in determining the performance. Of these properties, the compressive strength was the most important. Figure 4.11 shows the age-strength curve for the concrete used in the 6000 psi field cast beams.

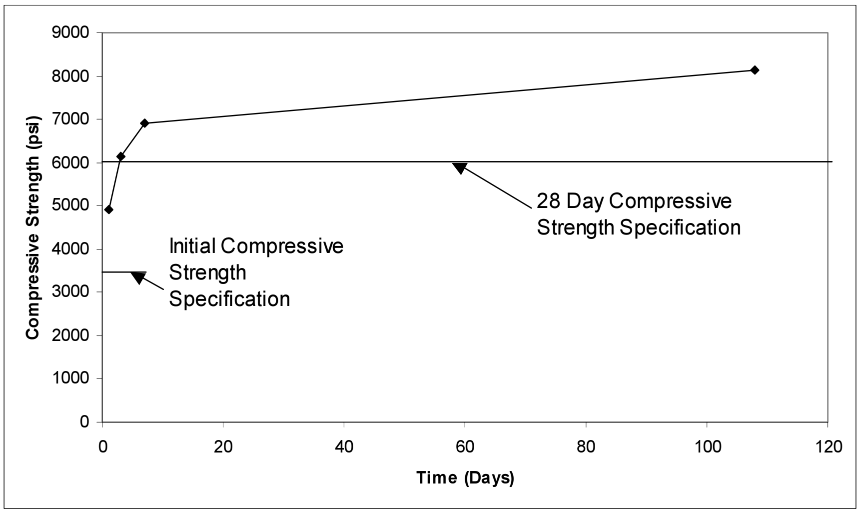

Figure 4.11 Age-Strength Curve of 6000 psi Mix

Interesting results arose out of the performance of the 6000 psi beams. The strengths were approximately 10 to 15% higher than the laboratory results, a surprising result since field results generally are less than laboratory results. The one-day strength was more than adequate for the prestressed yard as a compressive strength of 4950 psi was seen at release. The concrete continued to gain strength up to 8100 psi the day the first beam test took place.

This increased strength was a pleasant surprise. However, it did lead to the question of why there was so much extra strength. There was more cement in the mix, yet the proportions such as water-cement ratio stayed the same otherwise, so this would seem not to be the reason. Also, the compressive strength of this mix was much greater than the concrete used for the 20 ft beam which was produced at the same plant a couple of days earlier. Although a full age-strength curve was not obtained, the 1 day and 3 day strengths of that beam were measured as 3520 psi and 4629 psi, respectively. Since this mix was just used as a check, this data were used to proceed with the fabrication of the 40 foot beams.

No other reasons for the high strength can be proven or even hypothesized. A staff member closely watched the batching of the concrete and saw that the proportions measured were identical to the proportions ordered. Therefore, no additional cement was placed in the mix. The aggregate was in a moisture state between SSD and saturated, which was typical for mixing of the project.

The data presented at the beginning of this chapter should be taken as the typical mechanical properties of this mix. The compressive strength of the concrete at the precast plant was an aberration, although one that was positive.

4.3.3 Tensile Strength

Due to the time constraints of the project, 1 day splitting tensile and modulus of rupture tests were all that were taken. Figures 4.12 and 4.13 show their comparison to data already produced.

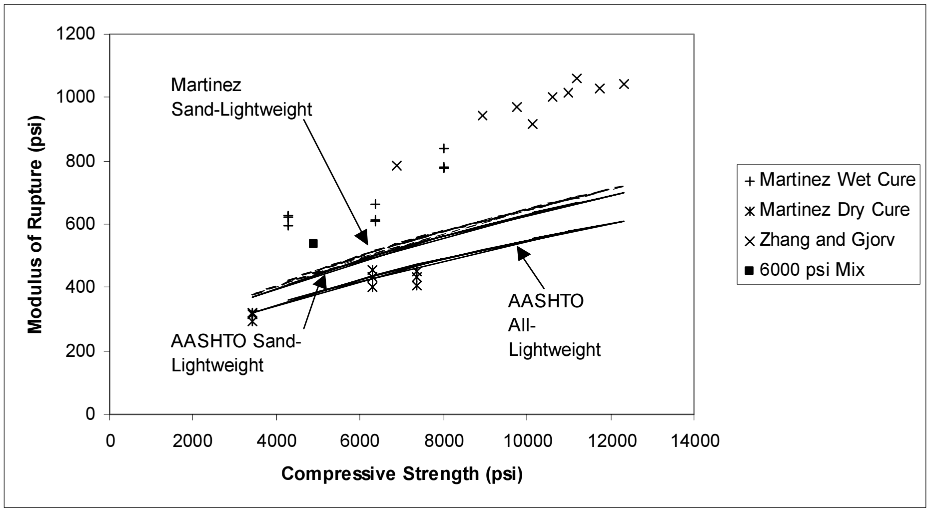

Figure 4.12 Comparison of MOR Data for 6000 psi Mix

From Figure 4.12, the 1 day 6000 psi MOR point exceeded both the AASHTO [1] and Martinez [50] recommendations. Although a 28 day data point was not taken, previous experience in the project assured the staff that the MOR at 28 days would be adequate since the concrete would still grow in strength.

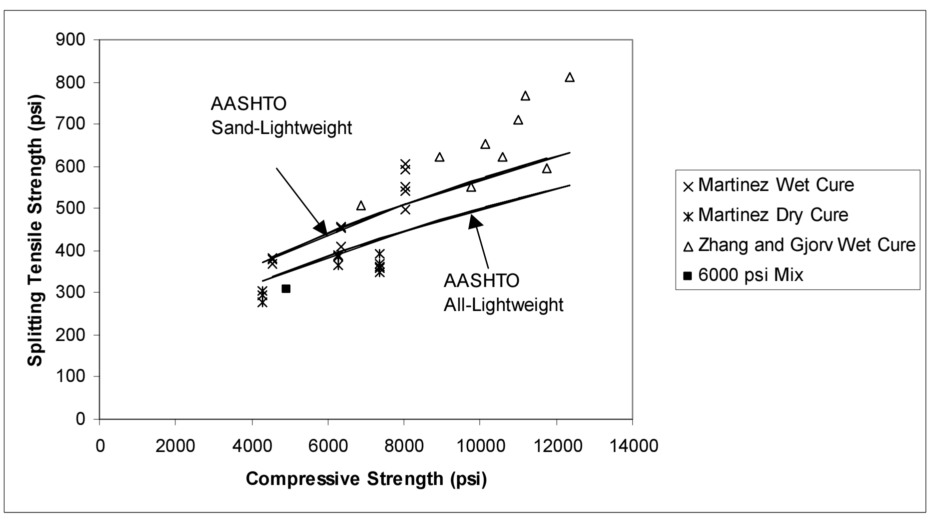

Figure 4.13 Comparison of Splitting Tensile Data for 6000 psi Mix

Just as in the last iteration, the splitting tensile data at 1 day was less than the AASHTO prediction. Although this occurred, the staff did not decide to do a 28 day test since the last iteration showed that the strength gain would be enough that the splitting tensile strength would exceed the AASHTO [1] equation at 28 days.

CHAPTER 5: NOMINAL 8000 PSI MIX FIELD TRIALS

5.1 MIX PROPORTIONS

As a companion to the 6000 psi mix, another high-strength concrete mix was developed. A higher strength, around 8000 psi, was desired to provide more options for applications for long span girders. A nominal 8000 psi mix was developed based on a previously designed normalweight mix from Myers [57]. To change this mix to a lightweight concrete mix, the coarse aggregate was replaced by an equal volume of lightweight aggregate.

Table 5.1 shows the mix proportions.

Table 5.1 Mix Proportions per Cubic Yard for 8000 psi Mix

Component	Proportion
Cement	671 lb
Fly Ash	316 lb
Lightweight Aggregate	1123 lb
Sand	1029 lb
Water	247 lb
Daratard-17	12 oz
ADVA Superflow	54 oz

A major difference from the 6000 psi mix discussed in the previous chapter was that this mix had 10.5 sack (987 pounds) of cementitious material. This is an extremely large amount. Problems associated with such high cement contents can include shrinkage cracks and high curing temperatures which can cause the concrete not to reach the target strength.

Also, the larger amount of cement meant that the aggregate amounts had to be reduced. Therefore, this mix was richer and less rocky than the 6000 psi mix. This also required an increased amount of superplasticizer.

5.2 PROPERTIES

5.2.1 Workability

Due to the large amount of cementitious material, workability was a major concern in the precasting yard. The high cement content dictated the use of a large amount of superplasticizer to produce the requisite flowing concrete that was needed for placement. Also, the elevated temperature of the concrete could substantially reduce the slump of the concrete. Because of these problems, the dosage of superplasticizer was an important aspect of this mix. The correct balance had to be obtained between an amount that provided for enough flowability and yet did not cause segregation of the concrete.

5.2.1.1 Slump

As in the 6000 psi mix, 7 to 9 inches of slump was the desired target. This was achieved by adding superplasticizer after an initial slump had been taken.

Two trial mixes were performed in the laboratory test phase to verify the performance of the mix. Both times, 10 fluid ounces of superplasticizer were added to achieve the appropriate amount of slump in 5 cubic foot trial batches. This comes to a dosage rate of 5.47 fluid ounces/100 pounds of cementitious material. During the laboratory trial period, the second mix had 9.5 inches of slump, two inches greater than the initial mix. This was a result of an increased retardant dosage, a different admixture from the superplasticizer. The dosage increased from 5 oz/cubic yard to 12 oz/ cubic yard of concrete. Since the retardant served as a water reducer, the slump increased.

One other aspect of slump noticed during the trial period was the loss of slump by the concrete. After the concrete was emptied out of the mixer, the concrete remained workable for approximately twenty minutes. After that time, the concrete grew increasingly difficult to work. When the final test beams and cylinders were being placed, approximately 30 minutes after removal from the mixer, the concrete only had about 3 to 4 inches of slump. This loss of slump was a concern. However, no modifications were made. Due to the speed at which concrete is usually placed at a precasting plant, the concrete was deemed be satisfactory.

5.2.1.2 Finishability

Again, finishability was not a major concern. Since this concrete would be going into beams, a rough finish was desired to promote bond between the slab and the beam. This bond also transfers horizontal shear between the two components of construction. The concrete contained too much coarse aggregate to achieve a flat surface on the top of the concrete not be satisfactory for deck concrete. The concrete was able to be placed into the forms and finished with a minimum of voids. This was the most important aspect.

5.2.2 Mechanical Properties

Obviously, mechanical properties again play the largest role in the acceptance of the concrete for use in pretensioning. The basic goal was 8000 psi at 28 days. A mix satisfying the 8000 psi goal, should more than satisfy the required 3500 psi 1 day strength for early release.

5.2.2.1 Compressive Strength

The results of the laboratory trials were very encouraging for use of this concrete. Figure 5.1 shows the age-strength relationship of the 8000 psi concrete mix.

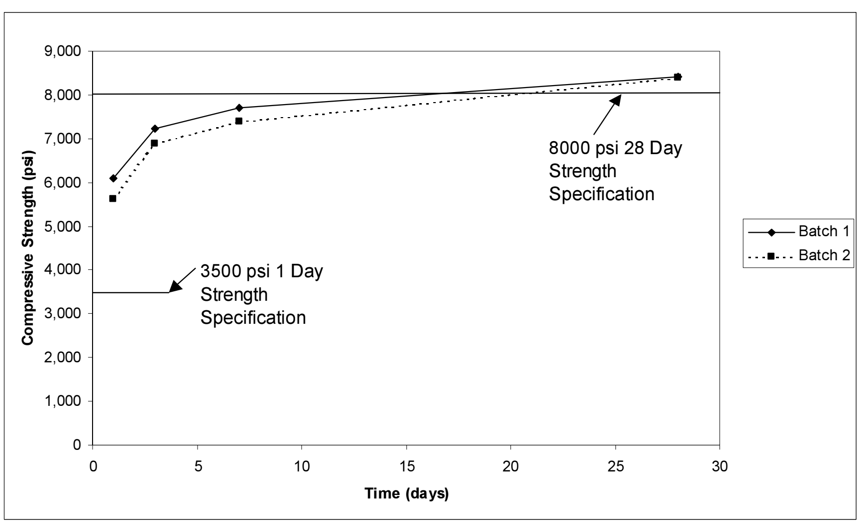

Figure 5.1 Age-Strength Relationship of 8000 psi Mix

As can be seen, the 8000 psi mix performed satisfactorily although with not as much margin as the 6000 psi mixes. Both batches achieved strengths in the mid 8000 psi range at 28 days. Also, the initial strength was far more than adequate with a strength of at least 5500 psi at one day.

The strength gain curves show that the concrete was continuing to gain strength at later ages. Also, while there was some significant early age difference between two concrete batches, at later ages, the strengths were remarkably similar.

5.2.2.2 Modulus of Elasticity

The 8000 psi concrete was expected to have somewhat larger values of modulus of elasticity than the 6000 psi concrete. Figure 5.2 shows the 6000 and 8000 psi mixes normalized on the same graph.

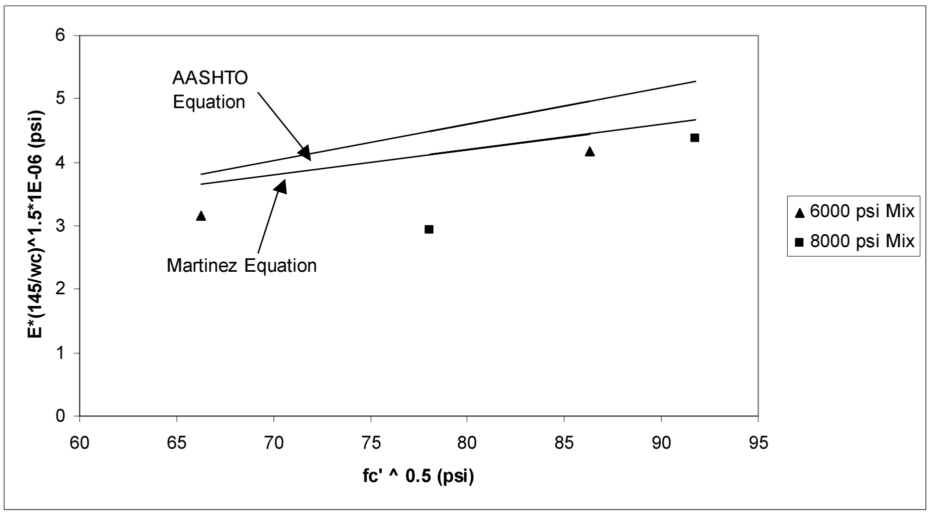

Figure 5.2 Relation of Moduli of Elasticity Values to Code Values

The two mixes produced in this project had moduli similar to the predictive equations, except for the 1-day 8000 psi value, which was quite low. In general, both the Martinez and AASHTO equations overestimated the modulus of elasticity values. However, there is always a great deal of scatter in these values. Figure 5.3 shows the 8000 psi data among many similar results.

The low value of the modulus does not indicate that the modulus of elasticity underperformed just for this concrete. It has been noted in other literature that the ACI-AASHTO equation overestimated the modulus of elasticity for high-strength lightweight concrete [50,80]. The Martinez [50] prediction was within 7%, which is fairly close when considering the inherent variability in modulus of elasticity measurements. Also, the 28-day value fell well within the scatter band produced by all the data, indicating that the 8000 psi mix has consistent performance with other high-strength lightweight concrete. Kolozs [43] goes into greater detail into the losses and deflections of the beams, which are the areas that are affected the most by the modulus of elasticity.

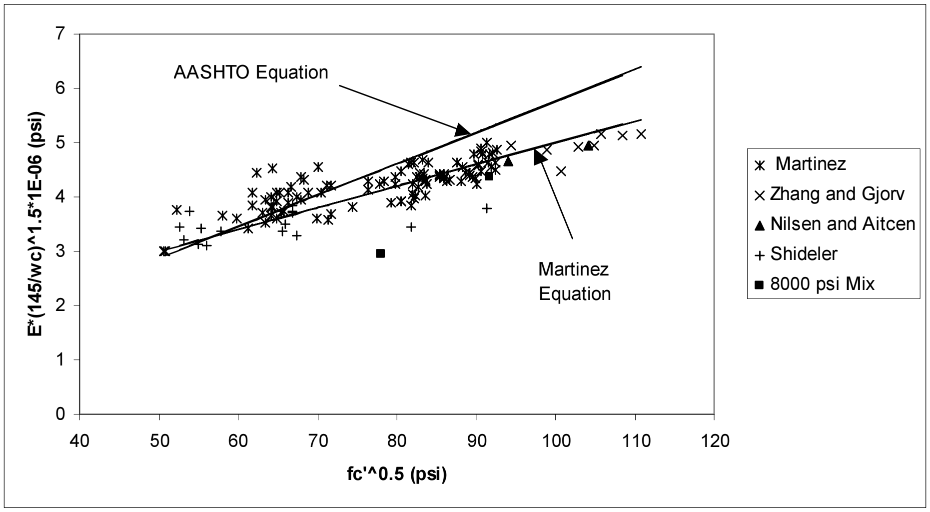

Figure 5.3 Comparison of Modulus of Elasticity for 8000 psi Concrete

5.2.2.3 Split Cylinder Tensile Strength

The split cylinder tensile strengths of the concrete again did not play a major role in the selection of the appropriate concrete mix. However, due to the use of tensile properties in prestressed design, the properties were measured to verify that the 8000 psi mix had adequate performance in this aspect. In terms of tensile strength, the 8000 psi mix actually performed relatively poorly. When compared to the 6000 psi mix, the values for the 8000 psi mix were much less. Splitting tensile strength of 6 X 12 inch cylinders was 318 psi at one day and 452 psi at 28 days. This was about 50 psi below those of the 6000 psi mix. Figure 5.4 gives a comparison of split tensile values to values from other studies.

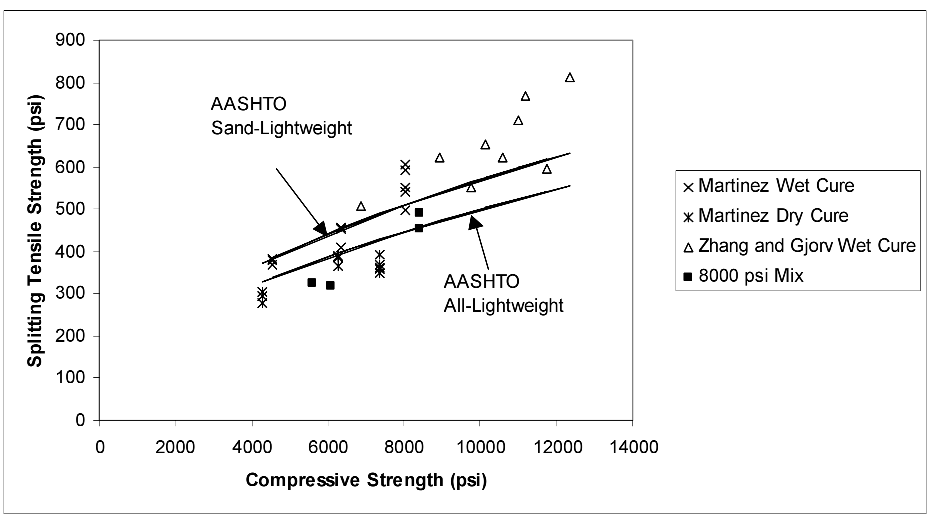

Figure 5.4 Comparison of 8000 psi Mix Split Tensile Values to Other Studies

Figure 5.4 shows that the values for the 8000 psi concrete were very low when compared to values from other studies. All four values fell beneath the equation given by AASHTO [1] for sand-lightweight concrete. The 28 day strengths, of more interest since these are the strengths usually given as the split tensile strengths of the concrete mix, are closer to the equation but still do not reach it. A possible reason to explain this discrepancy is the moisture state of the specimens when tested. The split cylinder test is very dependent on the moisture state of the specimen. Martinez [50] tested both wet and dry cured cylinders and found that the dry-cured cylinders had values 23% lower than those of corresponding moist-cured concretes. These concrete specimens had been allowed to dry for approximately an hour before testing. This could have caused the low values for the split cylinder test.

5.2.3 Creep and Shrinkage

5.2.3.1 Creep

The creep of the 8000 psi concrete mix was tested the same way as for the 6000 psi concrete mix. Details of the tests are given in Chapter 4.

Also, creep plus shrinkage again was measured in these tests since there was no control of humidity and temperature in the room where the creep tests were occurring. Therefore, the data are compared to data from Reichart [64], who compiled data on lightweight concrete behavior in creep and shrinkage.

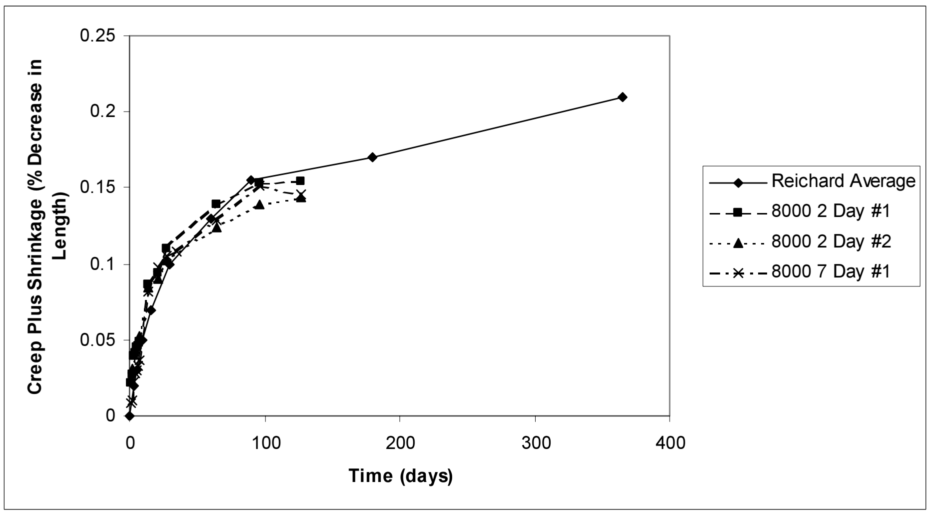

Figure 5.5 Creep Plus Shrinkage Behavior of 8000 psi Concrete Mix

Figure 5.5 shows that the 8000 psi concrete mix agrees very well with the Reichard [64] average for lightweight concrete in creep plus shrinkage. This shows that creep plus shrinkage of this concrete is normal.

Table 5.2 shows the creep plus shrinkage data in a different way. It utilizes Equation 4.1 and gives the measured creep coefficients.

Table 5.2 Four Month Creep Plus Shrinkage Behavior of 8000 psi Concrete

Age at Loading	Initial Elastic Strain (microstrain)	Creep Coefficient
2 days	725.4	3.13
2 days	764.6	2.88
7 days	750.6	2.95

Table 5.2 shows that the 8000 psi concrete mix behaves similarly to the 6000 psi concrete mix. Again, the creep coefficient averaged around 3, slightly less than the 6000 psi concrete mix. This can be attributed to the fact that the 8000 psi data were the four month data, not the five month data for the 6000 psi mix. At five months, the creep coefficient would probably be the same for both mixes.

Overall, the creep plus shrinkage behavior can be predicted using data from Reichard [64], which are the data in ACI Committee 213 [2] report. This result shows that the creep and shrinkage behavior of the 8000 psi mix is normal.

5.2.3.2 Shrinkage

Shrinkage also can affect long-term deformations of lightweight aggregate concrete. Figure 5.6 gives the shrinkage results for the 8000 psi concrete.

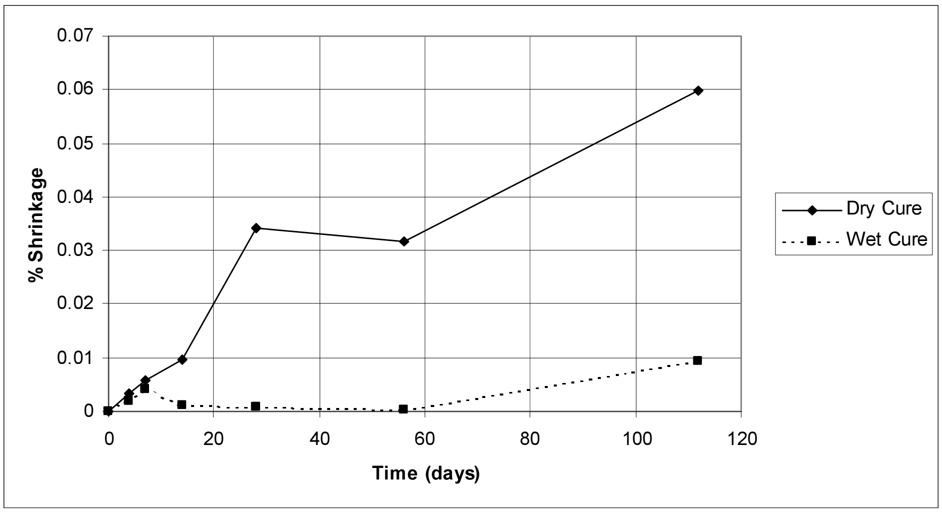

Figure 5.6 Shrinkage Results for 8000 psi Mix

Again, when compared to data from Reichard [64], the 8000 psi concrete proved to be average in its amount of shrinkage. Reichard showed that lightweight concrete decreased from 0.02% to 0.08% at 90 days due to drying shrinkage for dry cured specimens. From Figure 5.6, the 8000 psi concrete has decreased approximately 0.05% at 90 days, placing it firmly in the middle of data from Reichard. Also, Reichard [64] showed that lightweight concrete plateaus at approximately 150 days of age for drying shrinkage.

5.3 CASTING YARD PERFORMANCE

The performance of the 8000 psi mix at the casting yard did not measure up to the performance of the 6000 psi mix. Casting yard performance results were mixed.

5.3.1 Workability

More concern was always present when dealing with the 8000 psi mix. due to the high cement content and increased amount of superplasticizer used. In order to produce the three 40 foot concrete beams to be made of 8000 psi concrete, three batches of concrete were mixed. Each batch of concrete was three cubic yards in size. Again, 7-9 inches of slump was the target for this concrete. Each of the three batches of concrete used a different amount of superplasticizer. The results of workability tests are summarized in Table 5.3.

Figures 5.7 through 5.9 show the slumps of the concrete out in the field.

Table 5.3 Summary of Workability Results for 8000 psi Mix in Field

Batch	Superplasticizer Added	Dosage Rate	Slump at Batch Plant	Slump at Forms
1	215 oz	7.7 oz/100 wt	6.5 inches	3 inches
2	235 oz	8.4 oz/100 wt	8.5 inches	7 inches
3	265 oz	9.5 oz/100 wt	11 inches	8 inches

Figure 5.7 Slump for First Batch of 8000 psi Concrete

Figure 5.8 Slump for Second Batch of 8000 psi Concrete

Figure 5.9 Slump for Third Batch of 8000 psi Concrete

As can be seen, there was more need for adjusting superplasticizer for the 8000 psi mix. For each batch of concrete, the amount of superplasticizer was increased. The concrete was being sent out of the batch plant at adequate slumps. However, the concrete was losing slump during the short trip in the sidewinder. Most interesting of all was the last batch of concrete. The concrete was sent from the batch plant at 11 inches of slump, essentially flowing concrete. It behaved like water. By the time it had reached the line where the beams were being cast, it was down to the desired slump.

The workers at the precasting plant found the first two batches to be difficult to work and place into the appropriate forms when compared to the concrete mixes they used everyday at the precast plant. Unlike the 6000 psi concrete, a great deal of work had to be done to place the concrete into the forms.

Several reasons might have contributed to the problems with the loss of slump. First was the method of transportation used. The sidewinders did not have any way to mix the concrete during the trip. Therefore, the concrete sat unagitated in the sidewinder for 2-3 minutes until it reached the forms. Also, the concrete was vibrated by the trip since the sidewinder bounced along the road. This compacted the concrete. Instead of vibration, the concrete needed stirring to mix the contents instead of vibrating them.

Another reason probably better explains the slump loss. Since the mix is a 10.5 sack mix using Type III cement, the mix reaches extremely high temperature when compared to the more moderate-strength 6000 psi concrete. The high temperature of the concrete probably reduced the slump and caused problems with the workability. The day was already extremely hot with temperature around 95 degrees Fahrenheit. Although no temperature measurement was taken, the concrete was estimated at 110 degrees. This is a sign that the concrete was too hot and could lead to trouble.

Soroka and Ravina [73] documented this phenomenon. They showed that slump loss in concrete is accelerated by temperatures over 86°F. Although no temperature was taken of the concrete, the ambient air temperature that day was approximately 95°F, hot enough to aid this process.

Also, Punkki, et al. [63] showed that high-strength concrete utilizing a large amount of superplasticizer had problems with loss of workability. They showed that besides slump, which decreased, high-strength concrete showed a loss in plastic viscosity greater than the loss in normal strength concrete. This very likely is the explanation for the cohesiveness shown by the concrete.

However, these problems with placement did not cause any problems with the final side surface finish of the concrete. The completed prestressed bridge girders compared well with girders made with normalweight concrete. As shown in Figure 5.10, there were no voids or honeycombing which might be expected with a cohesive mix such as the 8000 psi concrete mix.

Figure 5.10 Finish of Girder with 8000 psi Concrete Mix

Overall, the workability performance was disappointing. Although the workers were able to place and vibrate the concrete into forms, the concrete was not as easy to work with as the 8000 psi laboratory concrete.

5.3.2 Compressive Strength

After the extraordinary performance of the 6000 psi mix, it was hoped that the 8000 psi mix would show enhanced compressive strengths. However, problems with workability indicated that the 8000 psi concrete fabricated at the plant might not perform similarly to the 8000 psi concrete fabricated in the laboratory.

Figure 5.11 gives the age-strength data for this mix.

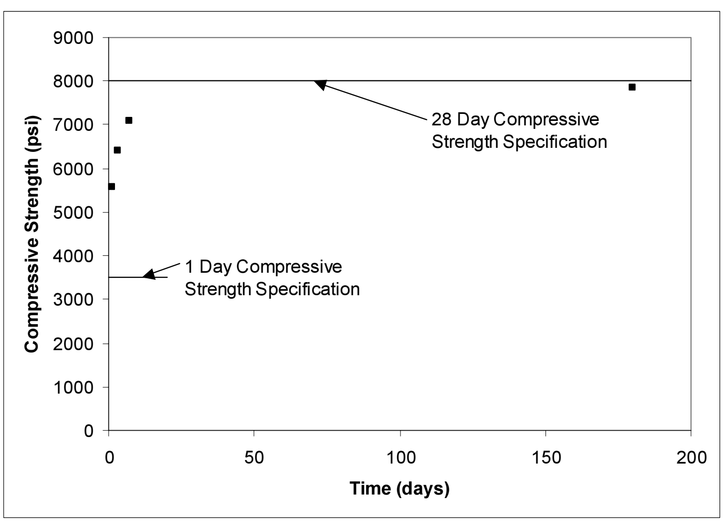

Figure 5.11 Age-Strength Curve for 8000 psi Concrete Used in Beams

In Figure 5.11, no curve is drawn between the points to approximate the strength gain. 28-day compressive strength was not measured due to a lack of cylinders. After the 7 day tests had been completed, the staff realized that not enough cylinders existed to allow 28-day compressive tests and also compressive and modulus of elasticity tests for the beam development length tests that would come later in the project. Therefore, a 180-day compressive strength test was done since a beam test took place at this date.

As can be seen from Figure 5.11, the 28-day strength was probably around 7500 psi and certainly was not adequate. Therefore, this mix did not meet the desired goal. It is believed that this occurred for the same reasons that workability was not adequate. With the slump loss often occurs loss in strength. It is not realistic to believe that this mix can be used at 8000 psi under plant conditions. 7500 psi is a much more realistic value and probably the practical upper limit for this project.

5.3.3 Flexural and Split Cylinder Tensile Strength

The 8000 psi concrete field mix also was tested at 1 day for flexural and split cylinder tensile strength. Figure 5.12 and 5.13 present this data and compare it to values from other studies.

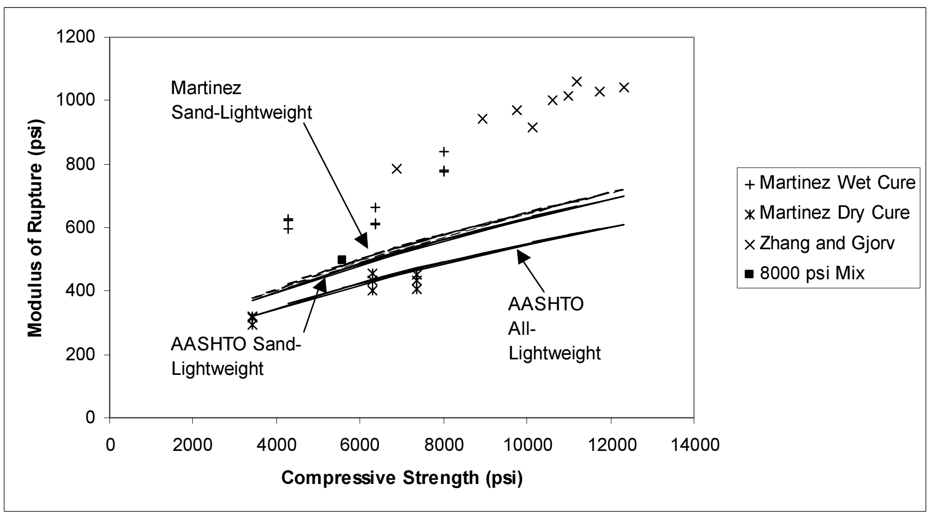

Figure 5.12 Flexural Strength of 8000 psi Mix Compared to Other Values

As seen in Figure 5.12, the 1-day flexural strength of the 8000 psi field mix was adequate. The value exceeded both the AASHTO [1] and Martinez [50] equations for flexural strength of sand-lightweight concrete.

As seen from Figure 5.13, the value of the 1-day splitting tensile strength fell beneath the AASHTO [1] equation value for sand-lightweight concrete. This result was similar to the earlier values for the split cylinder test for this same mix of concrete in the laboratory. Therefore, the likely reason for the low strength was the same, the drying out of the concrete before testing.

Overall, the performance of this concrete mix was disappointing. The 28-day strength goal was not met while the workability in the field was more difficult.

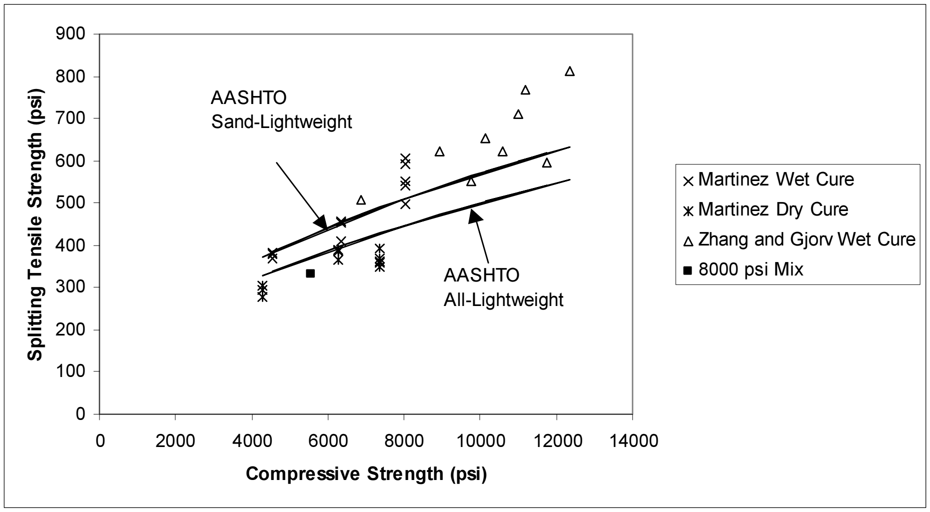

Figure 5.13 Splitting Tensile Strength of 8000 psi Mix Compared to Other Values

CHAPTER 6: CONCLUSIONS FOR THE MIX DESIGN PORTION OF THE PROJECT

6.1 SUMMARY

The initial portion of the project was carried out to determine the feasibility of developing usable high-strength lightweight concrete for possible use in pretensioned bridge girders. For this purpose, two distinct concrete mixes were developed. One was intended to have a 28-day strength of 6000 psi while the other was intended to have a 28-day strength of 8000 psi. Both also were intended to have a strength of at least 3500 psi at one day of age to expedite release of prestress in precast plants.

To obtain these two mixes, an ambitious laboratory mixing and testing program was implemented. Thirty-five mixes were designed and fabricated. For each of the mixes, mechanical behavior was determined to adequately document the concrete mix. Tests were performed to give compressive strength, modulus of elasticity, modulus of rupture, and splitting tensile strength. These four tests provide the most important data for utilization in girder design.

Furthermore, the slump of the concrete and finishability was noted for help in choosing the appropriate concrete. From these mixes, two laboratory concrete mixes were chosen that combined the best mechanical performance with adequate workability performance.

Using the 6000 psi mix, one 20 foot long and two 40 foot long beams were plant cast. Using the 8000 psi mix, one 20 foot long beam and three 40 foot long beams were similarly fabricated. The beams were then brought to Ferguson Structural Engineering Laboratory for performance testing.

6.2 CONCLUSIONS

6.2.1 6000 psi Mix

The 6000 psi mix was controlled by the 1 day strength requirement for the concrete. In order to achieve 3500 psi at 1 day with Type III cement and 25% replacement of fly ash, a certain minimum amount of cement was needed.

1) The one-day strength of the concrete was approximately 4000 psi.
2) The 28-day strength of the concrete averaged 7200 psi in the laboratory.
3) The 28-day strength of the concrete in the field was 7800 psi.
4) Approximately 5.5 fluid ounces of superplasticizer were required for every 100 pounds of cementitious material to produce the needed 7-9 inches of slump.
5) The fresh unit weight of the concrete was 127 pounds per cubic foot (pcf), which later decreased to 118 pcf at equilibrium conditions.
6) The concrete continued to gain some strength after 28 days.
7) The concrete completely filled the forms and the side finishes were very good at the precast plant.
8) This concrete mix provides about 30 minutes of working time under room temperature and average humidity conditions.
9) The concrete placed well at the precast plant. The workers could tell no difference between this mix and normalweight prestressed concrete girder mixes used at the plant.
10) Creep and shrinkage of the 6000 psi was high when compared to normalweight concrete. However, the results were reasonable when compared to other lightweight concrete.

The 6000 psi concrete mix produced excellent results. The concrete met all the mechanical strength requirements while also providing the needed workability.

6.2.2 Nominal 8000 psi Mix

The nominal 8000 psi mix was controlled by the 28 day strength of the concrete. To reach this goal, a very large amount of cement was required.

1) The 1-day strength was 5500 psi, easily surpassing the 1-day strength requirement.
2) The 28-day strength of this mix in the laboratory was 8600 psi.
3) The 180-day strength of this mix in the field was 7900 psi and the 28-day strength was probably only 7500 psi.
4) This concrete required a superplasticizer dosage of 7 fluid ounces per 100 pounds of cement.
5) The fresh unit weight of the concrete was 129 pcf, dropping to 122 pcf at equilibrium conditions.
6) The concrete was somewhat difficult to work in the laboratory; however, it was not unmanageable.
7) At room temperature and humidity conditions, this concrete only gives about 20 minutes of workability.
8) At the precast plant, the workers had trouble placing the concrete. They said that the 8000 psi mix required a lot of work to be placed properly in the girder forms. The final product was very satisfactory in appearance.
9) Creep and shrinkage of the nominal 8000 psi mix was good. Again, it was high compared to normalweight concrete. However, the performance was good compared to lightweight aggregate concrete.

Overall, the performance of the 8000 psi concrete was somewhat disappointing. The field performance did not agree with the laboratory performance. The performance in the field was not adequate for 8000 psi concrete. It would be acceptable for a nominal 7500 psi mix.

6.3 IMPLEMENTATION

From the results of this phase of the study, the following recommendations are made.

1) The 6000 psi mix is recommended for use in precast plants for high-strength lightweight concrete. Using this amount of cement, 7000 psi is the expected minimum 28 day strength. 7500 psi would be more typical of the long-term strength expected from this mix. This provides excellent margins for plant use. The mix is presented in Table 6.1.

Table 6.1 Recommended Mix Proportions for 6000 psi Mix

Component	Proportion
Cement	504 lb
Fly Ash	168 lb
Lightweight Aggregate	1264 lb
Sand	1149 lb
Water	222 lb
Daratard-17	12 oz
ADVA Superflow	34 oz

2) The nominal 8000 psi mix should rerated as a 7500 psi mix. The field performance of the 8000 psi mix was marginal. Table 6.2 shows the 8000 psi mix (rerated as a 7500 psi mix).

Table 6.2 Recommeded Mix Proportions for 7500 psi Mix

Component	Proportion
Cement	671 lb
Fly Ash	316 lb
Lightweight Aggregate	1123 lb
Sand	1029 lb
Water	247 lb
Daratard-17	12 oz
ADVA Superflow	54 oz

3) The 6000 psi mix is more workable, making it easier to use for precast plants. Precast plants should be able to use the 6000 psi mix design with a minimum of training in lightweight concrete.

4) The 7500 psi mix is more risky with respect to workability. It may cause problems for precast plants who choose to use it, particularly in hot weather. Its high slump loss and loss in workability combine to make it a more difficult mix to use. Its use is suggested for precast plants whose personnel have wide experience with high strength concrete containing a large amount of cement per cubic yard. This mix can be used with minimal problems. It just requires workers with experience to control it

6.4 Recommendations for Future Study

The following are recommended for future study if usage of lightweight concrete girders is expanded.

1) Further lab mixing of the recommended 6000 psi and 8000 psi mixes to gain an average age-strength curve with standard deviations.

2) Exploration of the use of silica fume in lightweight concrete. Silica fume has been proven to produce lightweight concrete with strengths in excess of 10,000 psi [80].

3) Testing of different types of sand with different gradations to understand the interaction of sand with this lightweight aggregate.

4) Further laboratory testing of lightweight aggregate concrete to understand overyielding and underyielding.

CHAPTER 7: TRANSFER AND DEVELOPMENT LENGTH BACKGROUND

7.1 INTRODUCTION

This chapter includes background information relating to the bond characteristics of prestressing strand with emphasis on lightweight concrete.

7.2 BACKGROUND

This project dealt with two main topics: the structural use of high performance lightweight concrete (HPLC) in bridge girders and more specifically, the bond behavior of prestressing strand in HPLC. While a broad review of the literature on these topics was carried out, only relevant studies are reviewed herein.

7.2.1 Lightweight Concrete

A discussion of the basic properties and behavior of high performance lightweight concrete was discussed in Chapter 2 and further detail is given by Heffington [33].

7.2.2 Bond Behavior

When a pretensioned concrete beam is fabricated without the aid of anchorages, the precompression forces that develop the initial compressive stress in the concrete at tendon release are transferred from the strands completely by the bond between the pretensioned strands and the concrete. Increased forces in the strand are required for full development of the ultimate tensile strength of the prestressing strand at flexural ultimate of the beam. These increases in strand force also depend on bond forces between strand and concrete. A bond failure occurs when the bond forces allow slip of the prestressing strands relative to the concrete before development of the strand tensile strength.

There are three bond mechanisms that act between the concrete and strand to transfer the stress in the steel to the concrete; the Hoyer effect, adhesion, and mechanical interlock.

The Hoyer effect is the tendency of the prestressing strand to increase in diameter as the stress in the strand decreases at release [35]. Initially, before casting of the concrete, when the strand is in open air and is being stressed to its initial pretensioning stress, f_{pi}, its diameter will decrease due to the Poisson's ratio effect. When the strand is released after the concrete cast around the pretensioned strands has cured, the strand not contained within the concrete will increase in diameter as it resumes its original size. However, as shown in Figure 7.1, the strand embedded in the concrete varies from zero stress at the very outer edge of the concrete to a constant stress at some distance located along the strand inside the concrete. This constant stress is known as the effective prestress, f_{se}, and the distance at which it occurs is the transfer length, L_t. Therefore, the maximum increase in strand diameter occurs at the inner edge of the concrete, where the stress is zero. The strand gradually decreases in diameter over the transfer length. The resulting taper in of the strand acts like a wedge, resisting the pull of the strand. The concrete resists this wedging effect, transferring part of the stress from the strand to the concrete. This mechanism is termed the Hoyer effect. The Hoyer effect is the greatest contributing mechanism to the bond at release of the initial prestress.

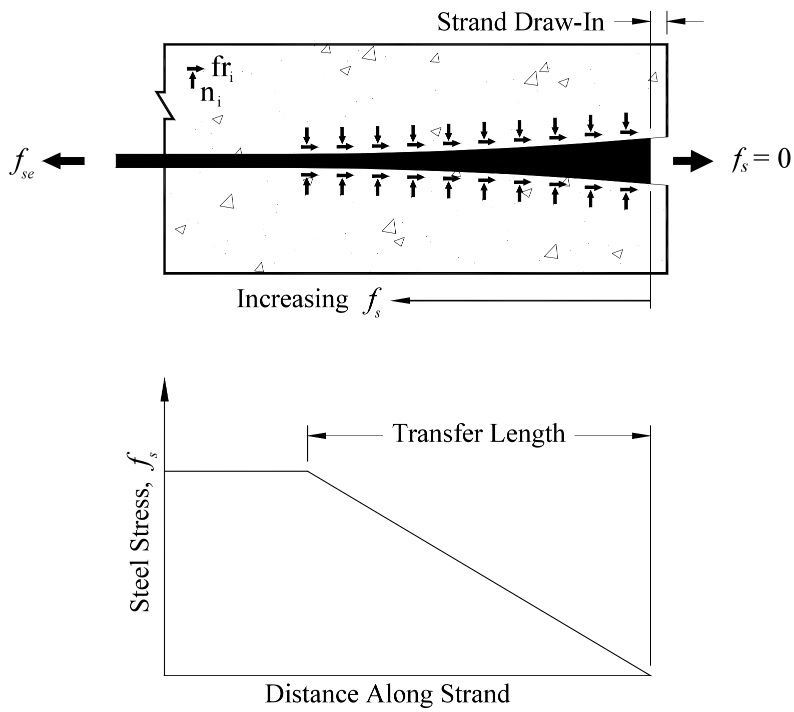

Figure 7.1 Diagram of Hoyer Effect [65]

Mechanical interlock is another mechanism by which the strand transfers stress to the concrete. This mechanism only occurs with twisted strand and is due to the tendency of the strand to want to unwind at increased stress levels [65]. Since the concrete cast around the strand has hardened, it conforms exactly to the stressed shape of the strand. Therefore, the concrete resists the unwinding or pull-out of the strand, increasing the amount of stress transferred to the concrete. Mechanical interlock is the main contributor to bond when the stress in the strand is increased above the initial transfer stresses. This occurs under loading when cracking of the concrete reaches the level of the strands. The resulting elongation accompanies increased stress in the strands, as the strands are prevented from unwinding by the surrounding concrete.

Adhesion is another mechanism that aids in transferring the stress in the prestressing steel to the concrete. It is the chemical mechanism by which the concrete bonds to the strands [65]. This bond acts only to a certain level. If the bond stress is greater than the critical adhesion stress, a local bond failure between the strand and concrete will occur. Adhesion aids bond transfer in areas where slip of the strand does not occur relative to the concrete. This mechanism is the smallest contributor to developing bond stresses between strand and the concrete.

7.2.2.1 Definitions

There are two types of bond stresses (transfer and flexural) and three types of bond lengths (transfer, flexural bond, and development) that describe the behavior of bond in prestressed concrete. The embedment length is a term that relates to the development length. All terms need careful definition:

Transfer bond stresses are the shear stresses developed on the concrete-steel strand interface due to bond mechanisms that result in the increase from zero stress in the steel and concrete at the free end of the member to the effective prestress in the steel at the transfer length. The main bond mechanism that contributes to this change in stress is the Hoyer effect [65].

Flexural bond stresses are the shear stresses developed on the concrete-steel interface due to bond mechanisms that increase the stress in the steel from the effective prestress before loading to the ultimate stress occurring in the strand, f_{pu}, as applied moment increases to the ultimate moment, M_u. The main bond mechanism that contributes to this change in stress is mechanical interlock.

There are different distances along the concrete member over which the transfer and flexural bond stresses act as shown in Figure 7.2:

Transfer length, L_t, is the distance from the end of the concrete member to the point where the stress in the prestressing steel becomes constant. This constant stress in the prestressing steel is referred to as the effective prestress after losses, f_{se}.

Flexural bond length, L_{fb}, is the distance from the end of the transfer length zone to a point at which the ultimate stress in the strand, f_{ps}, can be developed.

Development length, L_d, is the distance from the end of the concrete member to the point at which the ultimate tensile stress in the steel can be developed without bond failure. The development length is the sum of the transfer and flexural bond lengths.

Embedment length, L_e, is the actual bonded length of the tendon from where the stress is zero to a particular critical section. Depending on the applied load, the critical section usually occurs at the point of maximum moment. The embedment length is important because it defines whether a bond failure ($L_e < L_d$) or a flexural failure ($L_e > L_d$) will occur in the beam. A bond failure occurs when the strand slips relative to the concrete. Flexural failure occurs when the strand reaches its ultimate tensile strength before slipping.

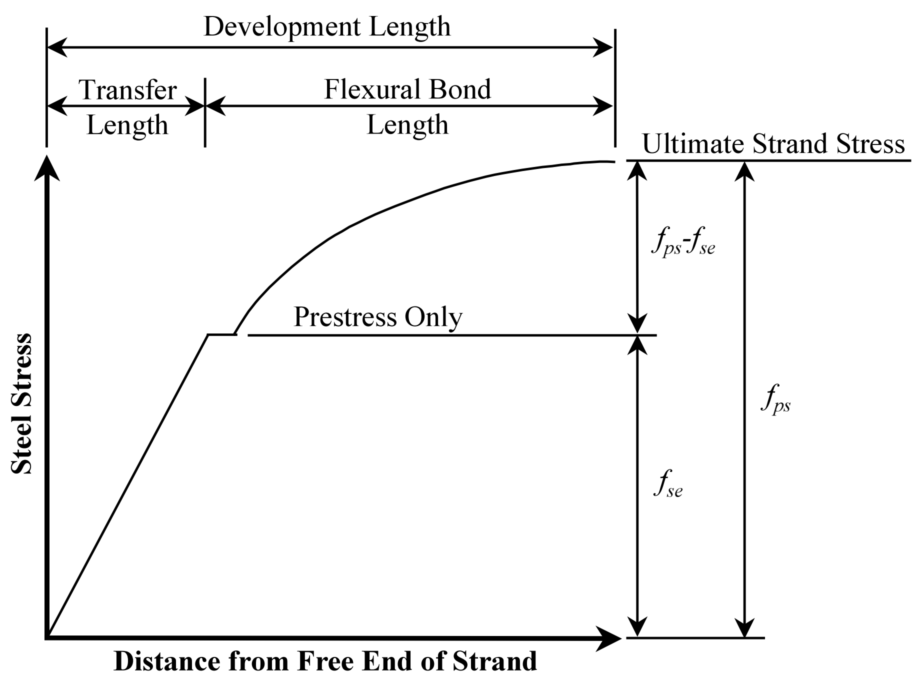

Figure 7.2 Variation of Steel Stress [3]

7.3 Literature Review

This section briefly summarizes previous literature regarding possible use and testing of normalweight and lightweight pretensioned concrete beams.

7.3.1 Use of Lightweight Concrete

7.3.1.1 Zia, P. – [84]

This study indicates that commercially available HSLC using natural sand fines was available in compressive strengths ranging from 5,000 – 7,000 psi. Also, by using fly ash and water reducing agents, the compressive strength could be increased to 8,000 – 10,000 psi.

Calculations were performed by Zia to determine the differences in the amount of prestressing strand and the necessary concrete compressive strength for various span bridges for normalweight or for lightweight concrete. For both short and long span bridges, Zia showed that with the same concrete compressive strength there was more than a 10% reduction in the amount of prestressing steel area required for completely lightweight concrete girder-slab bridges as compared to normalweight concrete bridges.

The study also indicates that for a given concrete compressive strength, the maximum span distance of the girders is greatest for lightweight concrete bridges, although a limiting factor is the deflection limit under service load.

7.3.1.2 Yang, Y. & Holm, T. – [79]

This paper reviews a T.Y. Lin report [27] and the progress made since its publication. The researchers found that replacement of normalweight concrete decks with lightweight concrete decks increased the live load capacity of bridges studied in the report and also allowed increased deck widths. It also indicates that the number of lightweight concrete bridges being built each year was increasing, with over 500 major bridges then completed in North America.

The economic advantages of using lightweight concrete in bridges were also discussed. For example, the paper indicates that the dead load from self-weight for long span concrete bridges can be 85% of the total load applied to the structure. The use of lightweight concrete in place of normalweight concrete could reduce the total applied load on the structure by 25%. This reduction in dead load would favorably impact the cost of the piers and foundation.

The paper also indicates that there may be concerns about deflections of bridges using lightweight concrete, which has a lower modulus of elasticity than normalweight concrete. The problems presented by deflection can be alleviated by:

1) Precambering
2) Providing additional cables to modify the stress diagram
3) Providing spaces for additional cables to be stressed one or two years after completion of the project
4) Using higher strength (say 30% higher) concrete for the bottom slab of the box section

7.3.1.3 Other Sources

The scope of this report does not include a complete review of the literature pertaining to the use of lightweight concrete. The few sources presented in this section are representative of the type of information that exists on the feasibility of using lightweight concrete in bridges. Other sources can be found in Heffington [33] and Thatcher [75].

7.3.2 Testing of Lightweight Concrete Beams

7.3.2.1 Ahmad, S. & Barker, R. – [4]

This study consisted of six single reinforced (bottom strands only) HSLC beams with varied concrete compressive strength from 5,200 to 11,000 psi, and with tensile steel content as a ratio of the balanced steel content (ρ/ρ_b) from 0.18 to 0.54. The average unit dry weight for the lightweight concrete was

122 lb/ft^3. The experimental results showed that the ultimate moment capacity exceeded the ACI predicted capacity by an average of 7%, with a maximum of 12%. The following two conclusion of this study are especially pertinent:

1) The ACI equivalent rectangular stress block provides accurate, conservative predictions of the ultimate flexural capacity of singly reinforced lightweight concrete members having a compressive strength not exceeding 11,000 psi and a ρ/ρ_b not exceeding 0.54.
2) The ACI recommendation of 0.003 as the maximum usable concrete strain appears to be an acceptable lower bound for HSLC members with compressive strength not exceeding 11,000 psi and ρ/ρ_b value less than 0.54.

7.3.2.2 Swamy, R. & Ibrahim, A. – [5]

This study examines the strength, cracking, and deformation characteristics of expanded slate (Solite) prestressed lightweight concrete beams. The cube compressive strength varied from 5,000 – 5,800 psi (34.5 – 40.0 MPa) and the unit weight varied from 105 – 108 lb/ft^3 (1,659 – 1,706 kg/m^3). The study found that the average ratio between the ultimate moment developed in the beam and the cracking moment was 1.70. They also found that the ultimate moment determined with Whitney's Rectangular-Stress Block overestimated the actual moment capacity by 5%. All failures for the beams were flexural in nature and ultimate capacity was reached by crushing of the compression concrete. The conclusions of this study were that the deflection under design loads of the Solite concrete was within parameters defined in the ACI Building Code. Also, the prestressed beams showed adequate ductility and deflection prior to failure. They found that the strain distribution in the lightweight concrete beams were similar to those for normalweight concrete beams. Finally, they observed that the failure zone of the lightweight concrete beams was more extensive than that of normalweight concrete beams.

7.3.2.3 Mor, A. – [55]

This research study was used to determine the effects of condensed silica fume on the mechanical bond properties when used in HSLC. Normalweight and lightweight concrete mixes were produced with and without adding silica fume. The lightweight mixes had densities between 126 – 130 lb/ft^3. All the mixes had 28-day compressive strengths above 9,000 psi.

Pullout tests were performed to determine the bond properties. The results of these tests led to the following conclusions:

1) Bond strength of normalweight concrete and lightweight concrete of similar high compressive strength without silica fume was similar.
2) Bond strength of HSLC with silica fume was about double that of every other concrete of the same compressive strength at a slip of 0.01 in 0.0254 mm).

7.3.3 Use of Deck Panels

7.3.3.1 Bieschke, L., and Klingner, R. - [18]

The purpose of this study was to determine the effect of transverse strand extensions on the behavior of precast pretensioned panel bridges. Full scale static and dynamic testing was performed at FSEL on a three-girder (AASTHO Type II) bridge incorporating panels with and without transverse strand extensions. Normalweight concrete was used in the casting of the panels and beams. Placement of the panels on the girders had details similar to those described in Chapter 8 of this report.

The study found that AASHTO punching shear theory gave conservative estimates of the failure load at interior bridge deck locations and that it underestimated the failure at the middle areas of the deck overhang. Flexural yield-line theory gave conservative results for the behavior in both areas.

As to the behavior of the panels with and without the transverse strand extensions, the study found that this did not affect the overall or local behavior of the bridge. More importantly, the use of the fiberboard strips between the panels and beam was found to be an important detail in the performance of the composite section. When the panel was constructed with fiberboard strips, the panel could bear on the concrete that flows into this area after it has hardened. This prevented longitudinal cracking of the panel that might have occurred if the panel bore on the fiberboard strip or concrete beam alone.

7.3.3.2 Barnoff, R. – [16]

This study discusses the use of precast prestressed concrete deck panels as a method of reducing the deterioration of concrete decks due to improper placement of the concrete, improper curing of the concrete, insufficient concrete cover over the reinforcing steel, excessive use of deicing agents, and cracking of the deck slab due to overloads. For this research project a bridge was constructed using removable wood forms, permanent steel forms, plain butt joints between adjacent panels, and keyed joints between adjacent panels. Design and overload load tests were then performed on the bridge.

The prestressed panels were constructed of normalweight concrete with a 3-in (76.2-mm) depth and were pretensioned with 7/16-in (11.1-mm) diameter 7-wire strand. Testing at service loads found that full composite action was developed between the beam and deck when panels were used. This composite action was also present at failure. Tests were also conducted on laboratory specimens.

The conclusions of the study were the following:

1) Bridge decks constructed with panels and a cast-in-place topping can be assumed to be continuous over the supporting beams.
2) The AASHTO formula used to compute moments in bridge decks is conservative when compared with experimental results measured from static loads.
3) The type of joint used between panels has no effect on the behavior of the deck.

7.3.4 Transfer and Development Length Studies

The behavior associated with the transfer and development length of prestressing strand in concrete is complex and dependent on many variables. The ACI 318 [3] and AASHTO [1] Codes along with many other sources only consider certain major factors affecting the transfer and development lengths. This section reviews the past and present literature pertaining to the models used for comparison of the test results in this research project.

7.3.4.1 Martin, L. & Scott, N. – [49]

The researchers in this study indicate that the values for transfer and development length in the ACI Code were determined from a study by Kaar and Magura [31]. All the tests that Kaar and Magura performed were on normalweight concrete specimens. The researchers contended that in the Kaar and Magura study only one test beam failed as a result of bond failure. Also, the equations developed from these tests were based on final bond failure instead of first bond failure. There is a significant increase in load carrying capacity between these two events after first bond failure (slip) due to the contribution of mechanical interlock between the prestressing strand and the surrounding concrete. They believe that first bond failure data should be used as an indication of development length bond failure.

They suggest that the transfer length should be equal to eighty times the strand diameter, d_b. For the development length they suggest that a bilinear curve be used to determine the calculated stress in the steel, f_{ps}. The equations they give for determining this value are:

$$L_e \le 80d_b \qquad f_{ps} \le \frac{L_e}{80d_b}\left(\frac{135}{d_b^{1/6}} + 31\right)$$ **Equation 7.1**

$$L_e \ge 80d_b \qquad f_{ps} \le \frac{135}{d_b^{1/6}} + \frac{0.39L_e}{d_b}$$ **Equation 7.2**

7.3.4.2 Mitchell, D., et. al. – [54]

This team of researchers examined the influence of concrete strength on the transfer and development length for pretensioned strand in normalweight concrete. They performed tests of 22 precast pretensioned beams, varying the concrete compressive strength from 3,050 – 7,050 psi at transfer and from 4,500 – 12,900 psi for development length testing. They also expressed concern that the equations developed by Kaar and Hanson were not accurate.

To determine the transfer length they used the slope-intercept method. They used the equation developed by Zia and Mostafa [83] as the model for their equation. They concluded that the transfer length should be a function of the initial stress in the steel, f_{si}, and concrete compressive strength at transfer, f'_{ci}. The square root factor in Eq. 7.3 (units in ksi) is used as a correction factor to account for the influence of concrete strength at transfer.

$$L_t = \frac{f_{si} d_b}{3}\sqrt{\frac{3}{f'_{ci}}}$$ **Equation 7.3**

Beam tests showed that an increase in concrete compressive strength, f_c', also corresponded with an increase in the flexural bond length. They determined that the flexural bond length was a function of the effective prestress in the steel and the ultimate concrete compressive strength. Based on their tests and the transfer length equation that they developed, they formulated an equation for the development length given below as Eq. 7.4 (units in ksi):

$$L_d = L_t + (f_{ps} - f_{se})d_b\sqrt{\frac{4.5}{f'_c}}$$ **Equation 7.4**

7.3.4.3 Russell, B. & Burns, N. – [66]

The researchers performed tests on 44 test specimens to determine transfer length. They also reviewed the results from past research involving transfer length studies. Their tests indicated that AASHTO type beam specimens had shorter transfer lengths than simple rectangular 1, 3, and 5 strand specimens. They also indicated that current codes do not take into account the effect of concrete strength on transfer length, while recent studies had shown that it was a factor. However, they said that an exact equation was not necessary for design of safe structures. Based on their testing and analysis of research data from several other research studies they developed an equation that would encompass all the values that had been presented in valid tests to give a safe approximation of the transfer length. This expression is given as Eq. 7.5 (units in ksi):

$$L_t = \frac{f_{se}}{2} d_b$$ **Equation 7.5**

7.3.4.4 Balazs, G. – [14]

A theoretical approach can be taken to find a correlation between the amount of draw-in at the free end of the strand, Δ_d, the initial strain in the strand, e_{si}, and the transfer length. The original equation formulated by Guyon is given as Eq. 7.6 [30].

$$L_t = \frac{\alpha \cdot \Delta_d}{\varepsilon_{si}}$$ **Equation 7.6**

In this equation, α is a term that accounts for either a constant load stress distribution, α=2, or a linear load stress distribution, α=3. Balazs states that Polish researchers had conducted tests which found this constant equal to 2.86 [84] while research performed by den Uijl obtained a value of 2.46 for α [6]. Balazs states that the problem with Guyon's equation is that the coefficient takes into account the assumed shape of the bond stress distribution. He develops an equation that did not have this problem (Eq. 7.7). Here the value of b is dependent on strand diameter. When b is equal to 1/3, Eq. 7.7 corresponds to α=3 in Eq. 7.6. When b=0, Eq. 7.7 corresponds to α=2 in Eq. 7.6. For ½-in diameter strand Balazs suggests a value for b equal to 1/2.

$$L_t = \frac{2}{1-b}\frac{\Delta_d}{\varepsilon_{si}}$$ **Equation 7.7**

Balazs also used the theory developed by Guyon and took into account the nonlinear nature of the phenomenon. Balazs offers two alternate equations (Eq. 7.8 & Eq. 7.9, units in mm & MPa) based on a nonlinear approach, where f'_{ci} in initial compressive strength of the concrete.

$$L_t = \frac{111\Delta_d^{0.625}}{f_{ci}^{'0.15}\varepsilon_{si}^{0.4}}$$ **Equation 7.8**

$$L_t = \frac{3.5 f_{si}}{\sqrt{f'_{ci}\sqrt{\Delta_d}}}$$ **Equation 7.9**

7.3.4.5 Zia, P. & Mostafa, T. – [83]

The researchers reviewed past studies, both theoretical and experimental, to determine the factors that affect transfer and flexural bond length. The factors they determined that affected transfer and flexural bond length were:

1) Type of steel (wire, strand)
2) Size of steel (diameter)
3) Steel stress level
4) Steel surface condition (i.e. clean or rusted)
5) Concrete compressive strength
6) Type of loading (static, repeated, impact)
7) Type of prestress release (gradual, sudden)
8) Confining reinforcement around steel (helix, stirrups)
9) Time-dependent effects
10) Consolidation and consistency of concrete around steel
11) Concrete cover around steel

Based on a review of previous experimental studies, they developed an equation that best approximated the transfer length. They indicated that the initial prestress, f_{si}, and not the effective prestress, f_{se}, was the important factor in determining the transfer length (Eq. 7.10). Also, the researchers believed that the initial compressive strength of the concrete at release was a factor.

$$L_t = 1.5\frac{f_{si}}{f'_{ci}}d_b - 4.6 \qquad \textbf{Equation 7.10}$$

To determine the flexural bond length the researchers looked at Hanson and Kaar's test data [31] and determined that the ultimate strength of the strands had occurred in beams at a shorter length before general bond failure occurred. They determined that the average bond stress was lower than what was implied by the ACI Code. They based their equation for flexural bond length on these facts. Combining this equation with the transfer length equation yielded the development length equation given as Eq. 7.11:

$$L_d = L_t + 1.25(f_{pu} - f_{se})d_b \qquad \textbf{Equation 7.11}$$

7.3.4.6 Buckner, C. – [21]

This report was commissioned by the FHWA to review the recent studies being completed at that time on transfer and development length, analyze the data, and formulate design equations and guidelines. The studies that Buckner examined were from such institutions as University of Tennessee at Knoxville, Florida Department of Transportation, University of Texas at Austin, Purdue University, Louisiana State University, and McGill University. He also reviewed past research. All the data for the study was for normalweight concrete.

For transfer length Buckner found that the largest factors affecting the transfer length was f_{si} and d_b, but Buckner also looked at individual contributions from other sources. When examining the correlation between the transfer length and the elastic modulus of the concrete on the data, Buckner formulated Eq. 7.12. This was a unique evaluation that had not been suggested in the past studies. The equation was developed to best fit the data with some variation expected due to the pooling of different research studies and the fact that all variables affecting transfer length could not be controlled. Because the elastic modulus of concrete, E_c, is a function of f_c' in normalweight concrete, his recommendation for transfer length reduced to an expression without the modulus as a factor. That expression is not reported here.

$$L_t = \frac{1250 f_{si} d_b}{E_c} \qquad \textbf{Equation 7.12}$$

Buckner also reviewed the available data on development length. He suggested that many researchers have proposed constant bond stress models in relation to flexural bond length, but that a linear bond stress model more accurately represented observed behavior. To incorporate this into his equation for development length, Buckner uses the variable λ and gives two equations for its determination. One is for general applications and the other for cases when strand stress at ultimate moment is approximated using Equation 18-3 of ACI 318 [3]. The equation he formulates is given as Eq. 7.13, where β_1 and ω_p are the same factors defined in ACI 318.

$$L_d = L_t + \lambda(f_{ps} - f_{se})d_b \qquad \textbf{Equation 7.13}$$

$$\lambda = \left(0.6 + 40\varepsilon_{ps}\right) \quad or \quad \left(0.72 + 0.102\frac{\beta_1}{\omega_p}\right)$$

where $(1.0 \le \lambda \le 2.0)$

7.3.4.7 Current ACI 318 and AASHTO Guidelines [3,1]

The ACI and AASHTO guidelines for transfer and development length use similar equations. The transfer length is first mentioned in ACI 318 in Chapter 11: Shear and Torsion. Here it states that the transfer length can be assumed to be 50 strand diameters. This is later discussed more fully in Chapter 12 of the

ACI Code where the development length is defined as the combination of the transfer and flexural bond lengths. The portion attributed to transfer is given as Eq. 7.14. The previous statement that the transfer length is equal to 50 diameters is based on this equation with f_{se} equal to 150 ksi.

$$L_t = \frac{f_{se}}{3} d_b \qquad \textbf{Equation 7.14}$$

The equation that ACI 318 uses to describe the development length is given in this excerpt from the code:

> 12.9.1 – Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length, in inches, not less than
>
> $$\left(f_{ps} - \frac{2}{3} f_{se} \right) d_b \qquad \textbf{Equation 7.15}$$
>
> where d_b is strand diameter in inches, and f_{ps} and f_{se} are expressed in kips/in^2. The expression in parenthesis is used as a constant without units.

In the commentary this equation is expanded to show the contribution of transfer and flexural bond length, which is Eq. 7.16:

$$L_d = \frac{f_{se}}{3} d_b + \left(f_{ps} - f_{se} \right) d_b \qquad \textbf{Equation 7.16}$$

The AASHTO LRFD Design Specifications [1] gives Eq. 7.15 for determination of the development length. Due to the higher levels of prestressing being used in construction, the transfer length has been increased in the new provisions. The AASHTO code indicates the "transfer length may be taken as 60 strand diameter."

CHAPTER 8: BEAM AND COMPONENT DESIGN AND CONSTRUCTION

8.1 INTRODUCTION

This chapter discusses the design, specification, and manufacture of the beams and components produced for the testing program. Since the project was a feasibility study of the use of pretensioned HPLC concrete bridge beams, the number of test beams commissioned by TxDOT was small. A total of eight beams with various combinations of concrete strength and types of concrete used in the beams and decks were fabricated.

8.1.1 Nomenclature

To facilitate understanding, each test will be referred to in the manner presented in Figure 8.1.

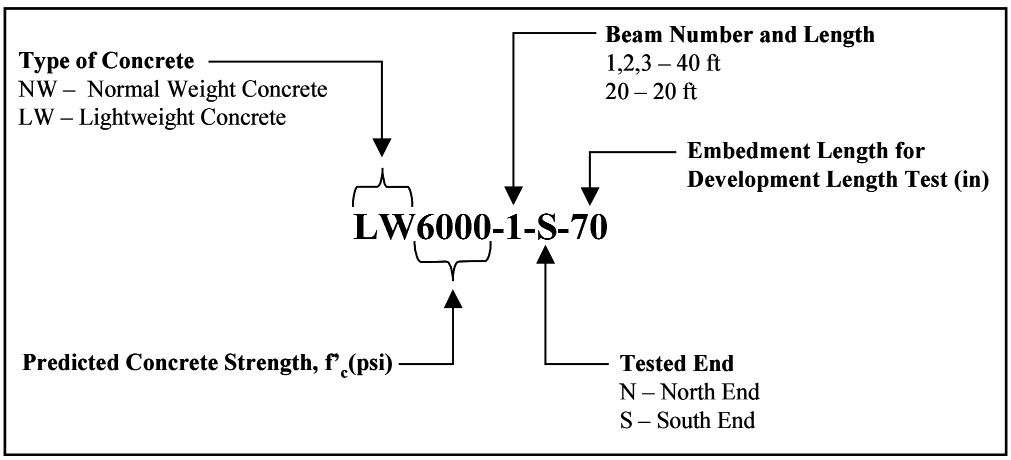

Figure 8.1 Beam Nomenclature

The full beam designation is not used at all times in the report because at times the reference in the text will be to a set of beams. For example, LW8000 will refer to the four lightweight 8,000-psi beams of various lengths and LW6000-20 will refer to one beam, 20-ft in length but including both ends of the beam. For transfer length tests, the reference NW6000-1-N will refer to one beam and one end of the beam. For the development length tests, the reference LW6000-1-N-80 will refer to one beam, one end, and the embedment length to be tested.

8.1.2 Number of Beams

A total of eight pretensioned AASHTO Type I beams were produced at Heldenfels Prestressing Plant in San Marcos, Texas. In addition to the six 40-ft long full size beams originally commissioned for the project, two 20-ft beams were also produced. The purpose of the reduced length 20-ft beams was to introduce the prestressing plant to lightweight concrete, identify any problems in the mix designs, ensure that all the facilities at the plant were in order to produce the actual test beams, and to examine placement problems in beams with maximum strand and reinforcement congestion. This was a valuable undertaking because several potential production problems were corrected and the subsequent fabrication of the six test beams occurred without any major problems.

The six 40-ft test beams were produced on two different days, in groups of three, due to the time constraints of instrumenting and measuring the beams after the forms were removed. One normalweight concrete beam and two lightweight concrete beams with a predicted 28-day nominal compressive strength of 6,000 psi were cast one day. Then the pretensioning force in the strands was released the next day. The other three lightweight concrete beams with a predicted 28-day nominal compressive strength of 8,000 psi were then produced in a similar way two days later.

8.1.3 Variables

The only property that was varied among the beams was the concrete mix design. This was done so that the measurements performed on each beam could be directly compared to the other beams and the type of concrete would be the only basis for difference among the results. A normalweight nominal 6,000-psi concrete mix obtained from Capital Aggregates was used for one beam (NW6000), which would be used as the control mix in the experiments. A lightweight nominal 6,000-psi concrete mix developed as part of this research project was used on two beams (LW6000). Finally, a lightweight nominal 8,000-psi concrete mix developed as part of this research project was used on the last three beams (LW8000). These variables are shown in Table 8.1, with further explanation of the deck properties given in the following sections.

Table 8.1 Test Beam Concrete Properties

Beam ID	Length	Beam Concrete	Deck Concrete	Lightweight Deck Panels
LW6000-20	20-ft	LW 6000-psi	None	N/A
LW8000-20	20-ft	LW 8000-psi	None	N/A
NW6000-1	40-ft	NW 6000-psi	NW 5000-psi	No
LW6000-1	40-ft	LW 6000-psi	NW 5000-psi	No
LW6000-2	40-ft	LW 6000-psi	NW 5000-psi	Yes
LW8000-1	40-ft	LW 8000-psi	NW 5000-psi	No
LW8000-2	40-ft	LW 8000-psi	NW 5000-psi	Yes
LW8000-3	40-ft	LW 8000-psi	LW 5000-psi	No

Notes:
LW = Lightweight
NW = Normalweight

8.2 DEVELOPMENT OF 20-FT DESIGNS

Several variables were tested by the inclusion of the two 20-ft beams in the test program. The beams were a shorter length than the other six beams that would be tested because they were not part of the original study. Their main purpose was to be a check of the production plants' ability to produce the lightweight concrete. One beam would be cast with the LW6000 mix design while the other beam used the LW8000 mix design. These were the same mixes that were later used for the 40-ft beams. To gain valuable information from the production of these beams, they were designed to test the lightweight concrete under the most congested strand pattern and highest level of prestressing conditions that might be applied in an actual AASHTO Type I bridge girder. This was determined by the TxDOT Bridge Design Division after running several computer models. The most heavily congested portion of the beam was at the ends, as shown in Figure 8.2. The typical dimensions of an AASHTO Type I beam and the strand pattern for these beams is shown in Figure 8.3. The strand number is given below each strand in the figure.

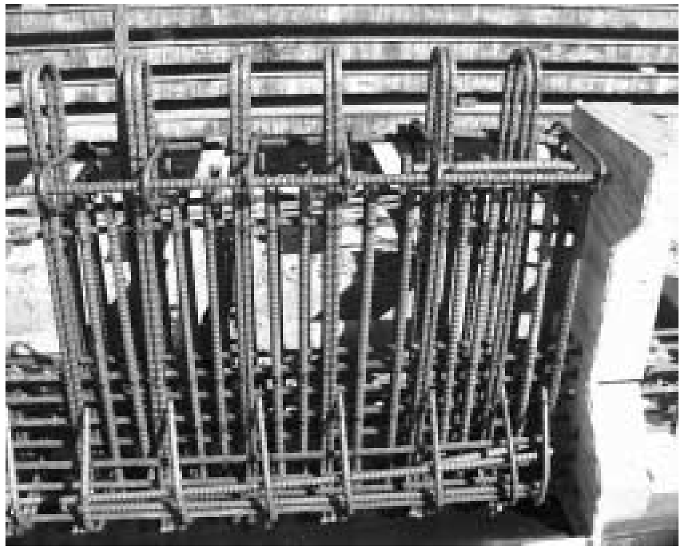

Figure 8.2 Congestion of Reinforcement at End of Beam

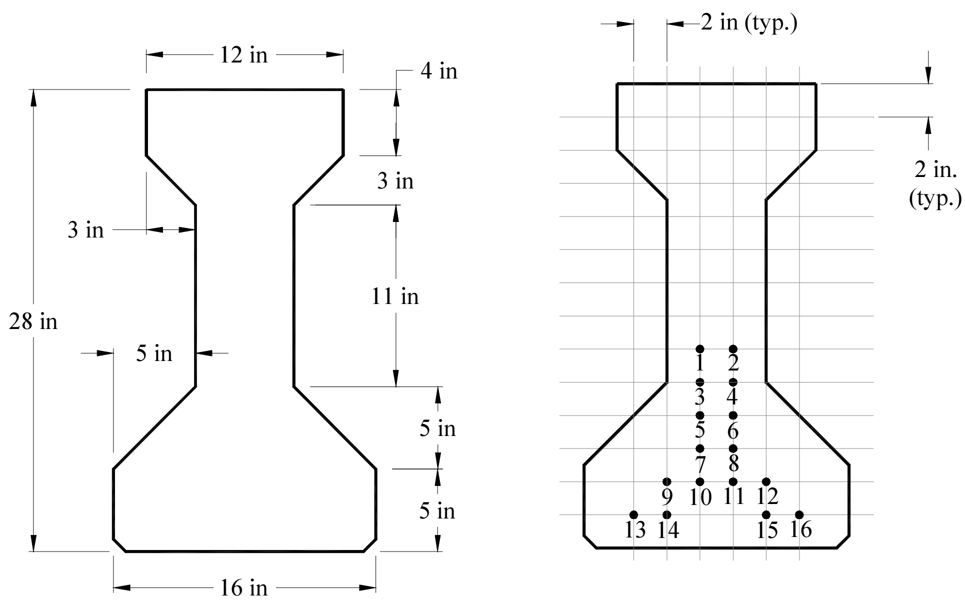

Figure 8.3 AASHTO Type I Cross Section and Strand Pattern for 20-ft Beams

These beams were also to be used as a teaching aid for the research personnel in applying the measurement devices since many of the people working on this phase of the project had no prior experience with the equipment. The data obtained would also give some initial data to help assess the adequacy of the amount of instrumentation required. Also, with the combination of expected lower tensile strength of the lightweight concrete and the maximum number of strands in the beam, these beams would show if end splitting cracking or cracking at the top of the member would occur. Neither of these effects was predicted by calculations and in fact did not occur in the actual beams

8.3 DEVELOPMENT OF 40-FT DESIGNS

The use of a 40-ft length for the test beams was determined by their effectiveness in past studies [15]. This allowed two tests to be performed on one beam without the damage from one test unduly influencing the next test. By using one beam for two tests it was possible to control the independent variables of the tests while knowing that the dependent variables, based on material properties, were the same. This was especially advantageous during development length testing where two different embedment lengths could be tested to help determine the development length.

The 40-ft beams used 12 strands placed in the pattern illustrated by Figure 8.4, with the strand number shown below each strand. This pattern is typical of a pattern that might be used in construction. Two strands were placed in the top flange to ensure that tension did not occur in this area, which might result in unwanted cracking. The bottom 10 strands were placed on the bottom two rows to give the largest possible moment arm for flexural resistance.

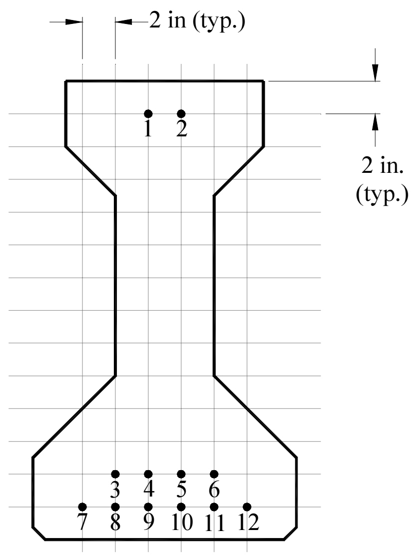

Figure 8.4 40-ft Beam, 12 Strand Pattern

The passive reinforcement used in the beams was determined by successful patterns used in similar tests, and by nominal TxDOT requirements. The amount of shear reinforcement used was more than would traditionally be used in a beam of this size. This was due to the fact that the selected shear span for the tests was between 60 – 80 in. Therefore, higher than normal shears would be produced in the beam. The design of the shear reinforcement was based on preventing shear failure in the beam at the ultimate load. A shear failure of the beam would not give any valuable data concerning the development length of the strands. From shear calculations, it was determined that double #4 stirrups were required in the end 10-ft of the beam to safely prevent this type of failure. Details of the reinforcement can be seen in Figures 8.5 and 8.6.

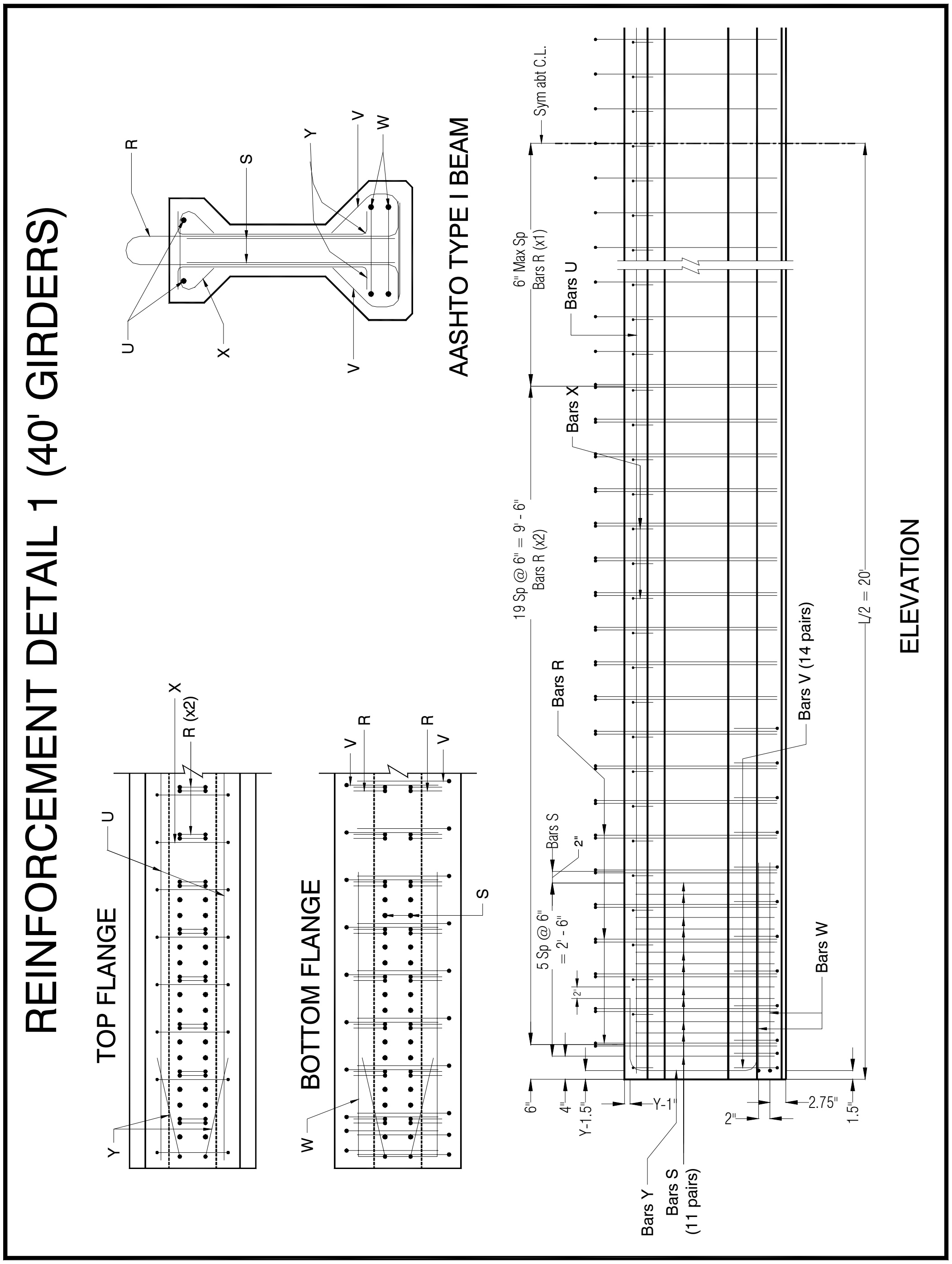

Figure 8.5 40-ft Test Beam Reinforcement Details

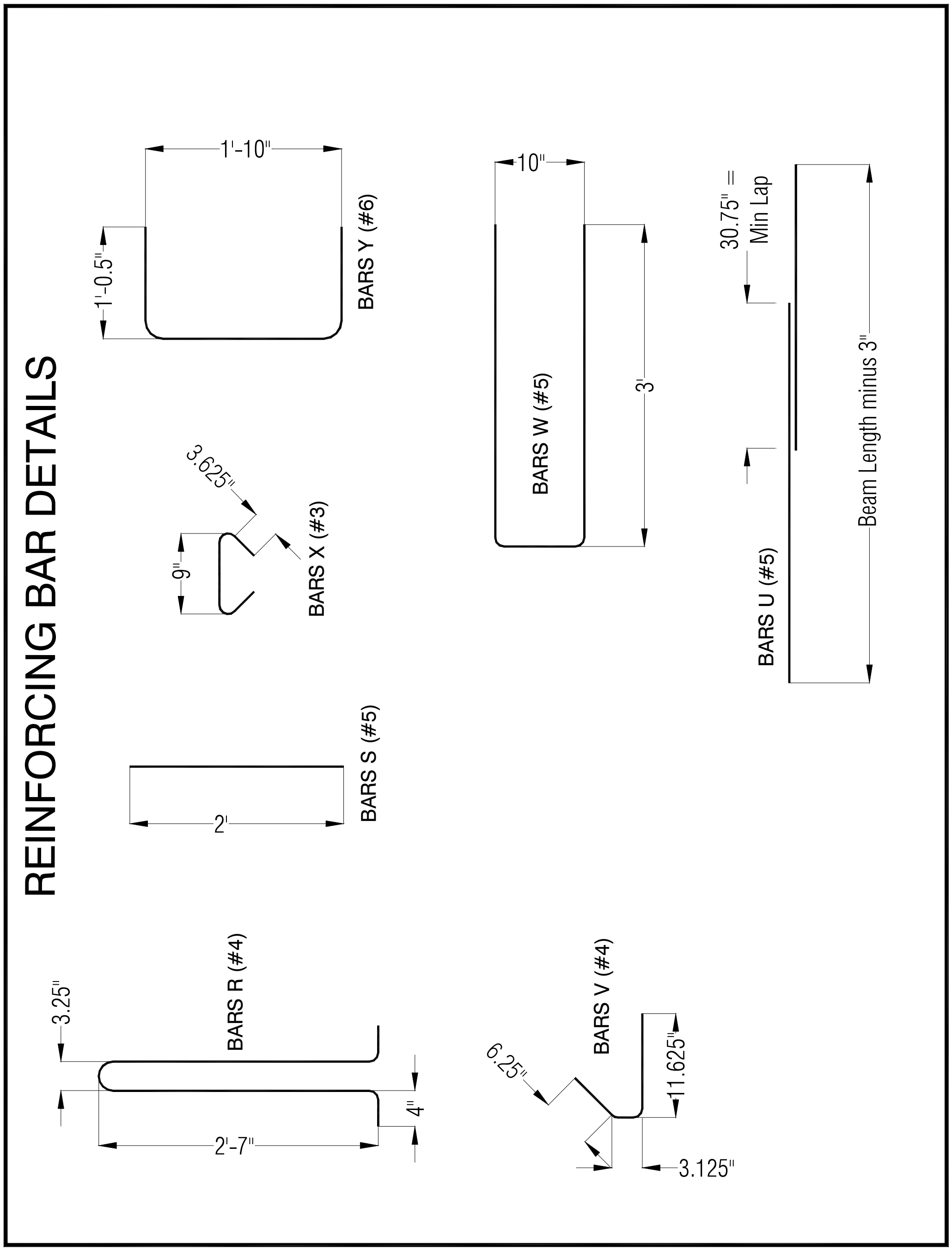

Figure 8.6 Reinforcing Bar Details

To perform development length testing on beams that accurately modeled a real bridge girder, a reinforced concrete deck was added at the Ferguson Structural Engineering Laboratory (FSEL). The type of concrete and deck details varied between beams as shown in Table 8.1. However, other details such as the depth and width of the slab and the amount of reinforcement were kept constant. The normalweight and lightweight decks that were cast used reinforcement details similar to those used in Texas bridges. The amount of reinforcement was determined by AASHTO code provisions [1]. The reinforcement details of the deck are shown in Figure 8.7. When the lightweight deck panels were used, a different reinforcement detail had to be used, as explained in the following section.

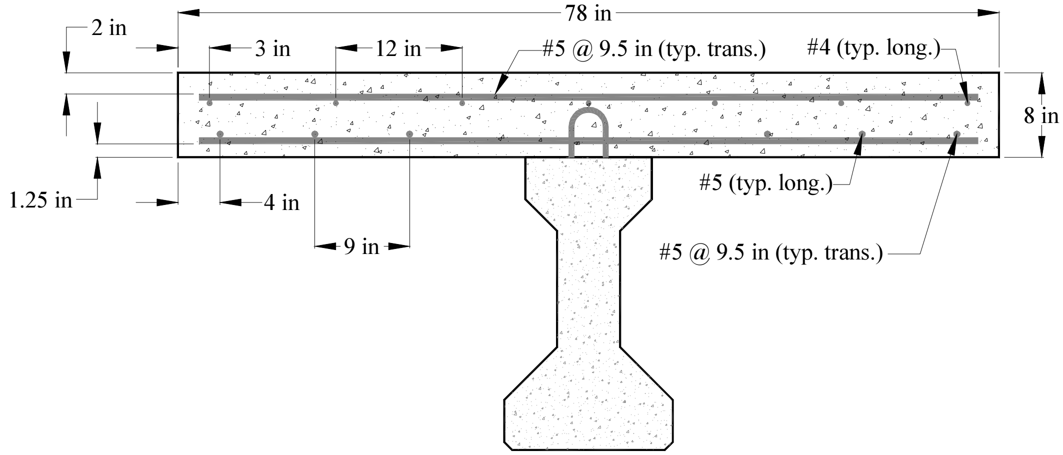

Figure 8.7 Normal and Lightweight Concrete Deck Details

8.4 LIGHTWEIGHT DECK PANEL DESIGN

Stay in place normalweight concrete deck panels are frequently used in construction to simplify the amount of formwork needed in the field. Typically, the panels are pretensioned with 3/8-in strands at 190 ksi. These panels are manufactured at a plant and therefore the quality is higher than would be expected in the field. The panels are then shipped to the bridge site for installation. The panels span the gap between beams and minimize the formwork required for the casting of the deck. The panels create a stable and safe working surface for support of the top layer of deck steel.

The initial proposal for this study did not include any deck panels. Possible savings by use of lightweight concrete in plant produced deck panels combined with normalweight cast-in-place deck completion was felt to be a promising alternative as the project developed. This avoided concern over lightweight concrete deck finishing and wear characteristics. It was proposed that lightweight panels should be included in the study. This was approved by TxDOT. The reason for exploring the use of lightweight deck panels is that the weight of the concrete in most bridges is typically the most significant load to which the bridge is subjected. By using lightweight panels, the dead load on the bridge can be reduced. This allows larger span lengths or wider spacing of the girders. This was a concept that TxDOT was interested in exploring. The panels were manufactured at Austin Prestress and the concrete provided by Rainbow Materials, who had extensive experience with lightweight concrete. Actual casting is shown in Figure 8.8.

In the field, a panel would span the distance between two beams. Since this study involved single beams with composite decks of a fixed width, a system had to be devised which would simulate field conditions. The interior edge of the panel was supported on the beam and did not require any modification from standard practices. In this case, the panel was supported on the edge of the beam by a 1-in wide by ½-in thick layer of fiberboard. This detail was used to provide more area to which the cast-in-place concrete could bond to the beam and develop composite action. This detail was also used because cracking problems have occurred in panels where the panel rested directly on the beam [18]. Three inches of the ends of the strands in the transverse direction of the panel were left exposed. This created dowel action on the free end of the panels to help support the load when the formwork was removed. These details are illustrated in Figure 8.9.

Figure 8.8 Lightweight Concrete Panel Casting

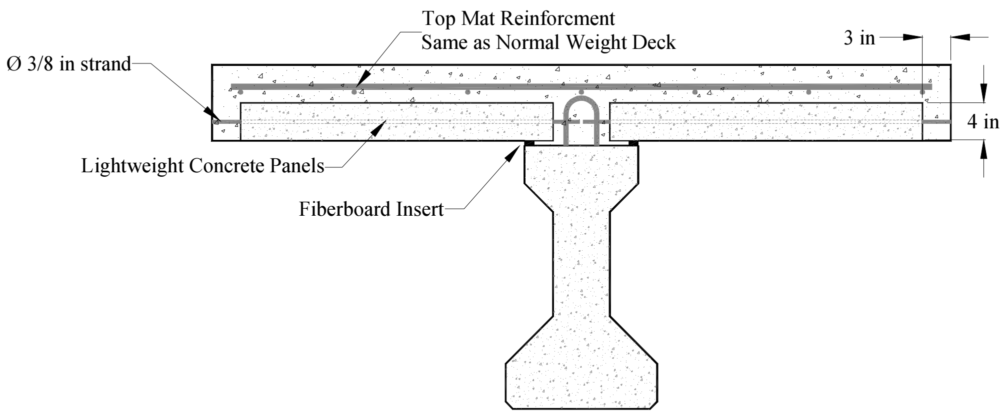

Figure 8.9 Lightweight Deck Panel Details

The actual cross section of the beam is shown in Figure 8.10. This shows the effectiveness of the fibrous material in creating a space for concrete to fill, adding to the shear capacity and bond between the beam and deck. After the cast-in-place deck concrete had cured, peepholes were drilled into the fiberboard every 4-ft to ensure that concrete had penetrated this area. No areas were found that had not been filled with concrete.

Two different panel sizes had to be manufactured to fit the 40-ft deck length. The layout of the panels is shown in Figure 8.11. The panels labeled "A" had extra reinforcement exposed on their ends so that steel would extend to the end of the beam.

Figure 8.10 Beam and Deck Cross-Section with Panels

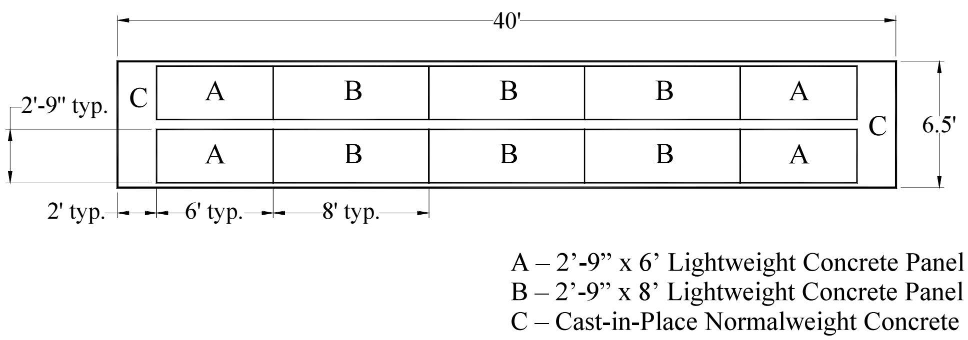

Figure 8.11 Top View of Deck with Lightweight Panel Layout

8.5 MATERIAL PROPERTIES

8.5.1 Concrete

The concrete mix designs were developed at FSEL and reported on in detail in Chapters 4 and 5. Three different concrete mixes were used in the production of the prestressed beams. The specified final compressive strength of the NW6000, LW6000, and LW8000 beams were 6,000 psi, 6,000 psi, and 8,000 psi, respectively. All the mixes were required to have a 3,500-psi release strength at one day. The release strength presented one problem for the mix design of the 6,000-psi lightweight concrete. No mix that was developed for this research study had a one-day release strength of 3,500-psi and a 28-day compressive strength of 6,000-psi. When mix designs were selected for the two lightweight concrete mixes, all mix data obtained by this study was analyzed. Two mixes that had 3,500-psi one-day release strengths and a difference of 1,000-psi in their 28-day strength were selected. The final strengths of the lightweight concretes did not vary as much as expected at the time of the development length tests. The mix designs for the normalweight and both lightweight mixes are given in Table 8.2. A summary of the relevant compressive strength properties, along with the unit weights, for all three concrete mixes used in this study are given in Table 8.3.

Table 8.2 Mix Designs

Material	NW6000	LW6000	LW8000
Water	250	222	247
Cement (Type III)	517	504	671
Fly Ash	0	168	316
3/4 in Hard Rock Coarse Aggregate	1869	0	0
3/4 in Lightweight Coarse Aggregate	0	1264	1123
Sand	1355	1149	1029
Retarder	12	12	12
Superplasticizer	20.4	34	54

Notes:
1) Quantities in lbs and oz
2) Quantities per yd^3

Table 8.3 Concrete Properties

Beam ID	Compressive Strength (f'_c), psi / (MPa)		Unit Weight lb/ft^3 / (kN/m^3)
	1-day	Long Term	
NW6000	3,490 (24.1)	5,500 (37.9)	149 (23.4)
LW6000	4,900 (33.8)	8,130 (56.1)	118 (18.5)
LW8000	5,560 (38.3)	7,850 (54.1)	122 (19.2)

8.5.2 Prestressing Steel

The prestressed steel reinforcement used in the test beams for this study was supplied by American Spring Wire Corporation. It was specified as ½-in, low-relaxation, ASTM A416, Grade 270 ksi steel. Mill certificates indicated an ultimate stress of 270 ksi and an elastic modulus of 28,000 ksi.

8.5.3 Reinforcing Steel

Mild steel reinforcement used in the test beams for this study was supplied by Border Steel. It was specified as Grade 60 steel. Mill certificates indicated a yield stress of 66 ksi and an ultimate stress of 100 ksi.

8.6 FORMING, PLACEMENT, AND CURING OF CONCRETE

The forming used at Heldenfel's Prestressing Plant for the beams was all- metal forms along the sides and bottoms of the beam. Both metal and wood forms were used as end forms, depending on what was available at the plant. The metal end forms tended to leave a better finish to the concrete, but were more difficult to remove due to their stiffness. The transport and placing of the concrete was performed by a sidewinder truck. Two lifts of concrete were placed for each beam because this allowed better consolidation of the concrete as it was placed. Hand-held vibrators were used to consolidate and remove air bubbles from the concrete. No external vibration was applied to the side forms. The finishing of the top of the beam was done by trowels. The surface was left rough so that a better bond could be established between the beam and deck when the deck was cast. After finishing, heavy mat blankets soaked in water were placed on top of the beams to minimize water loss. The beams were allowed to cure for one day until the cylinder compressive strength was near 3,500 psi.

The decks of the beams were all cast at FSEL. The forms used were all- wood. Placement of the concrete was accomplished by overhead bucket, and bull floats were used to finish the top surface of the concrete. Creating a very smooth finish was not necessary, so trowels were not used after bull floating. The finish of the concrete was adequate to allow a small area to be ground down and strain gauges placed. Plastic sheeting covered the entire unprotected surface of the concrete. Water was applied to the top of the deck while it cured to prevent water loss. The decks were allowed to cure for four days before the forms were removed.

CHAPTER 9: TRANSFER LENGTH TESTING

9.1 INTRODUCTION

The forms were removed from the concrete girders the day after casting. Prior to release of the strands, instrumentation was placed on the beams to determine the transfer length. Two different types of instrumentation were placed on the beams to obtain data on the transfer length. One method, DEMEC Strain Measurement, involved measurement of the strain in the concrete along the beam face. The other method, strand draw-in, measured the distance that the strand moved into the face of the concrete at the end of the beam. This chapter details the different types of instrumentation and measurement as well as the results and discussion of the transfer length testing for both beams and precast deck panels.

9.2 INSTRUMENTATION AND MEASUREMENT FOR DETERMINING TRANSFER LENGTH OF BEAMS

This section describes instrumentation and measurement techniques used on the beams for transfer length testing. The two methods described are DEMEC strain measurement and strand draw-in.

9.2.1 DEMEC Strain Measurements

The transfer length was previously defined as the distance from the end of the concrete member to the point where the prestressing steel stress becomes constant. Theoretically, for a beam with straight tendons, the force in the concrete beyond this point becomes constant. Therefore, the prestressing moment exerted on the beam also is constant. This moment develops a linear strain distribution in the cross-section of the beam that remains constant along the length of the beam at any distance beyond the transfer length. The strain in the concrete can be measured and the data used to locate where the strain becomes constant, and hence used to determine the transfer length.

The system used to measure the strain in the concrete for this project was the DEtachable MEChanical (DEMEC) Strain Measurement System. The DEMEC System involves gluing small metallic discs, ¼-in (6.4-mm) in diameter, to the face of the concrete spaced at the gauge length of the DEMEC gauge. Initially, the distance between the discs is measured before the strands are released to give an initial reading. This is the zero stress state of the concrete. The strands are then released and another measurement is taken of the distance between the discs. The difference between this reading and the previous one is the strain in the concrete and is measured directly by the DEMEC gauge. The accuracy of this device is ±25 microstrain [65]. This system is shown being used at the pretensioning plant in Figure 9.1.

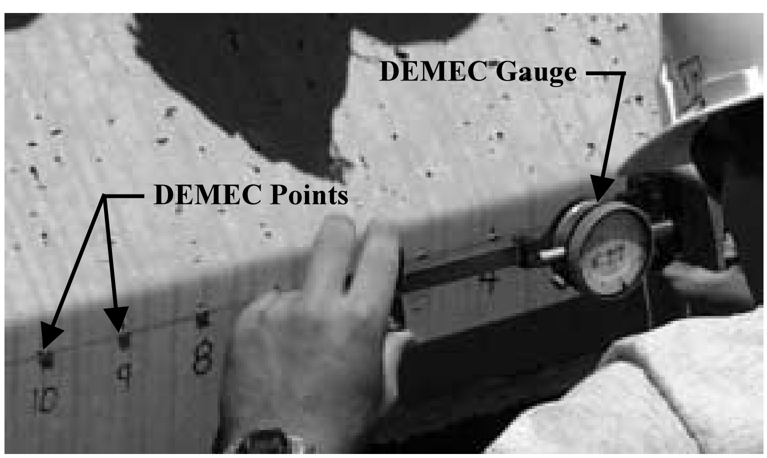

Figure 9.1 DEMEC Points and Measurement of Concrete Strains

It was determined from past use of this system that a spacing of 1.97 in (50 mm) would provide enough data to give a smooth strain profile [15,40]. The spacing and layout of the DEMEC points is shown in Figure 9.2. The distance that the DEMEC points extended along the beam was determined from calculations of the theoretical transfer length. For the pair of 20-ft beams, the DEMEC points were placed to a distance of 60 in from the end of the beam, or 30 DEMEC points. After reviewing the data and determining that a longer constant strain plateau was desired for the subsequent beams, the distance was increased to 79.7 in, or 40 DEMEC points. The gauge length of the DEMEC gauge used was 7.87 in (200 mm) and therefore 36 readings could be obtained from each set of DEMEC points. Each beam was instrumented with points on both sides of the beam. Therefore, each beam had two lines of points on its north and south end, corresponding to its east and west face, equaling a total of four lines of DEMEC points per beam. DEMEC points were placed on both sides of the beam so that variation of the strain in that area could be averaged using the readings from both sides of the beam.

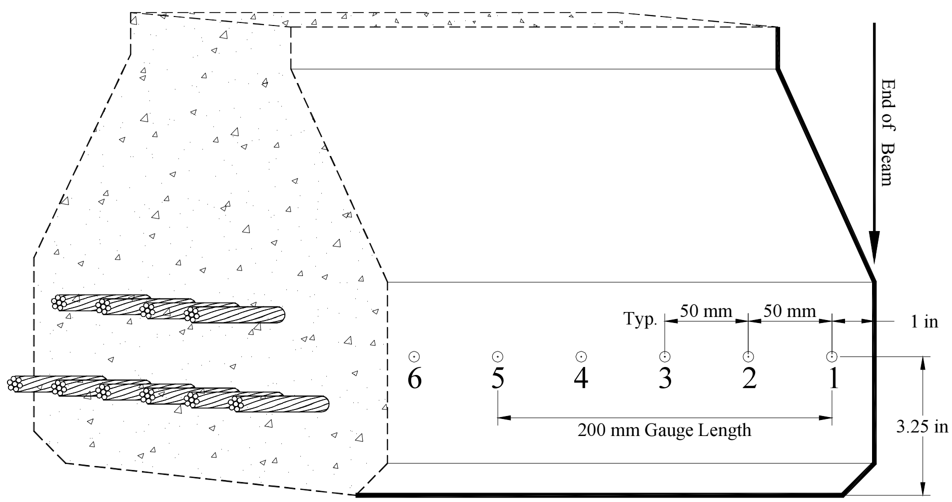

Figure 9.2 Spacing and Layout of DEMEC Points

9.2.2 Draw-In Measurements

Another method to determine the effectiveness of the bond between the concrete of the beam and the prestressing strand is to measure the draw-in of the strand after the prestress is released. The draw-in is a measurement of how far the strand at the face of the concrete is pulled into the beam after the prestress is released. Other terminology for this phenomenon besides draw-in is "suck-in" or "free end slip" [15]. The draw-in measurement can also be correlated with the transfer length of the strand since they both involve the bonding of the concrete to the strand [14,30].

Draw-in measurements were performed at the plant, one day after the beams were cast. A 4-ft (1.21-m) space had to be allotted between the ends of the beams because instrumentation had to be placed on the strands before release. This allowed removal of the end forms and enough working space to place the instrumentation and measure the draw-in values.

To measure the draw-in, aluminum U-channel measuring 1in x 1in x 1in and cut to a ½-in length was attached to each of the strands with a metal hose clamp. This method had been used successfully in past studies [15,40]. Plastic zip ties were used on some of the 20-ft specimens to test which system worked better. Use of the metal hose clamps, while being more time consuming to install, was found to be a more effective method and therefore these were used on all 40-ft specimens. A typical setup of one of these devices attached to a strand is shown in Figure 9.3. The surface of the concrete that would act as a datum for measurement was rough. Therefore, a glass slide was glued to the concrete at this location to provide a smooth surface.

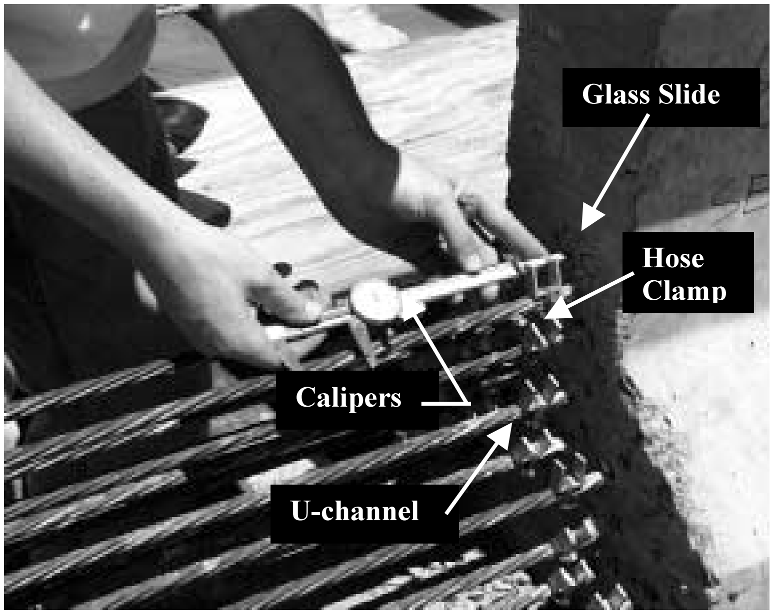

Figure 9.3 Instrumentation and Measurement of Strand Draw-In

To measure the draw-in, two measurements of the distance between the end of the channel and the face of the concrete had to be taken. The first measurement was performed before the pretensioning was released. After release another measurement was taken. All measurements were performed with analog calipers as shown in Figure 9.3. The point of contact of the end of the calipers and glass slide was marked with permanent marker along with the orientation of the calipers. This ensured that when the measurement was performed the second time, error would not be introduced by measuring a different point or orientation on the face of the beam.

The intended method of release of the pretensioning in the stands was by hydraulic jacking followed by gradual release of the pressure. This is the method typically used at the pretensioning plant. It would also facilitate measurement of draw-in because the strands would have little tendency to unwind and cause movement of the channel that was not associated with strand draw-in. The other method of release sometimes used is flame cutting of the strands. This instantaneously releases the prestress and there is a tendency for the twisted strand to unwind and ruin any opportunity for measurement. Unfortunately, due to communication problems at the plant, the top strands of beam LW6000-2 and LW6000-3 were flame cut and there was some unwinding of strand as shown in Figure 9.4. This did not seem to affect the readings of those strands because some of the prestress had been released and the hose clamps stopped the unwinding from extending into the area between the face of the concrete and the hose clamp.

Figure 9.4 Flame Cut Strands and Unwinding of Ends

9.3 DATA REDUCTION

The data from both the DEMEC strain and the strand draw-in measurements had to be reduced so that it could be used to determine the transfer lengths of the beams. This section describes these operations and the significance of the reductions.

9.3.1 DEMEC Strain Profile Smoothing

The value obtained from the measurement of two DEMEC points 7.87 in (200 mm) apart is applied to the middle of these two points. The 36 values (40 DEMEC Points) obtained from measuring one strip of points could be used to give a profile of the strain along the concrete, but this would not fully utilize the spacing of the DEMEC points. Some points overlapped the area in between the measurement of two points. The mid-point of these points falls in the area of the measurement of the original two points. These points include information about the strain in this area and therefore were also used. This technique is called smoothing the data [65] and is illustrated in Figure 9.5. The procedure takes three consecutive values and applies their average to the middle of these points. The general equation for this technique is given in Equation 9.1 [65].

$$\varepsilon_{i,smoothed} = \frac{\varepsilon_{i-1} + \varepsilon_i + \varepsilon_{i+1}}{3} \qquad \textbf{Equation 9.1}$$

The data obtained from the measurement of DEMEC points tends to have quite a bit of scatter associated with it. This is due to irregularity in the elastic modulus of the concrete at one-day combined with inherent variability in the DEMEC system. The smoothing technique will lessen this scatter and reduce the effect of data points that have values higher or lower than the average. By smoothing the data it is easier to define the plateau at which the constant strain in the beam is established.

The other operation necessary to reduce the data is to average the strain profiles for both sides of the beam. This will remove irregularities in the DEMEC data that only occurred on one side of the beam and accounts for differential heating of the beam due to sun exposure. The combination of both techniques allows a much more precise establishment of the constant strain plateau. A plot of the smoothed and averaged data from one end of a beam is compared to the raw data in Figure 9.6.

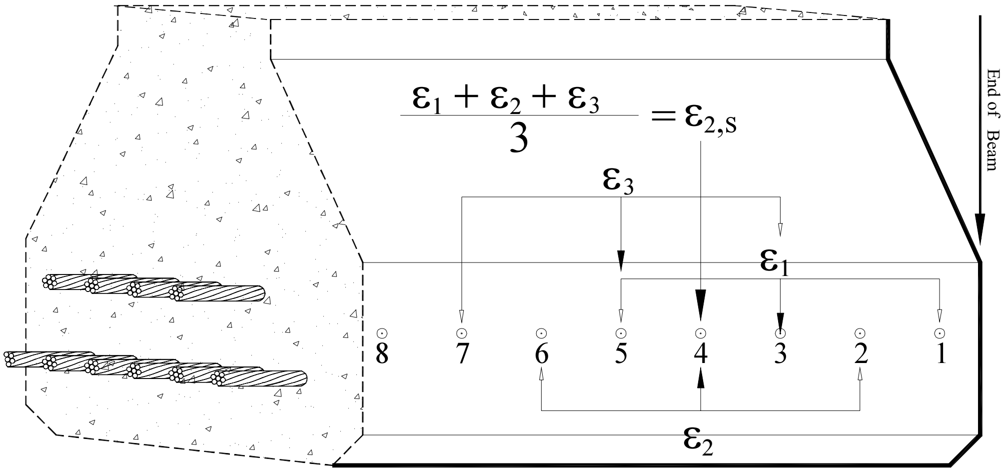

Figure 9.5 DEMEC Strain Profile Smoothing

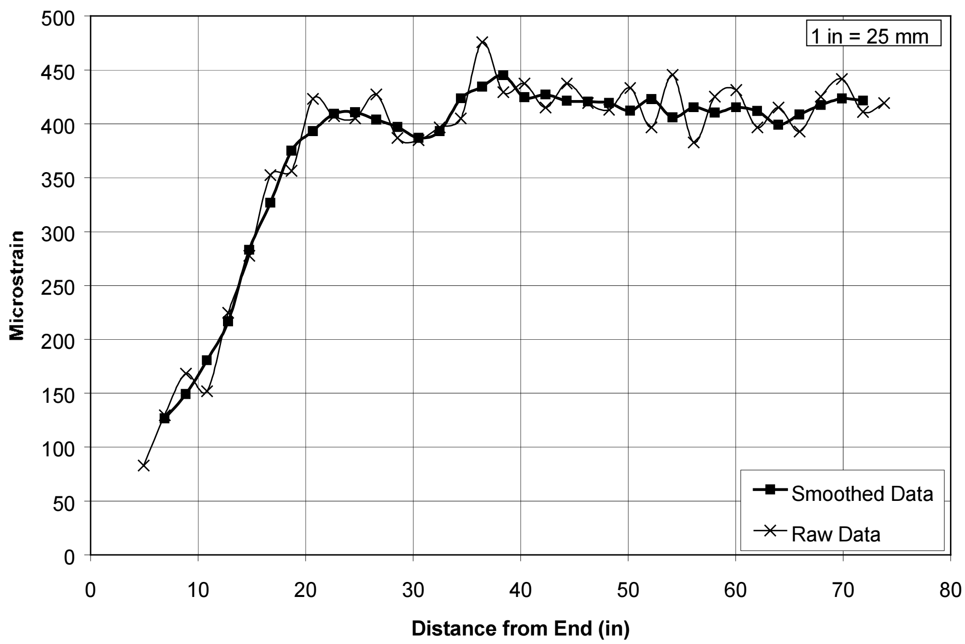

Figure 9.6 Strain Data Smoothing

9.3.2 Effect of Elastic Shortening on Draw-In Data

The difference in the values recorded from the measurement of the distance between the concrete and the end of the U-channel under prestressed and released conditions is not the actual draw-in of the strand. Since the strand is initially under a high stress, elastic elongation of the strand occurs. Therefore, this elongation must be subtracted from the total displacement measured to arrive at the actual draw-in of the strand. Figure 9.7 shows the measured value before release, L_i, the measured value after release, L_r, the difference between these values, Δ_t, which is a combination of elastic shortening, Δ_e, and the actual draw-in, Δ_d. Using these values and the strain in the steel under the prestressed conditions, ε_{si}, it is possible to formulate Equation 9.2 that describes the actual draw-in of the strand after release.

$$\Delta d = \frac{\Delta t - \varepsilon_{si} \cdot L_r}{1 + \varepsilon_{si}}$$ **Equation 9.2**

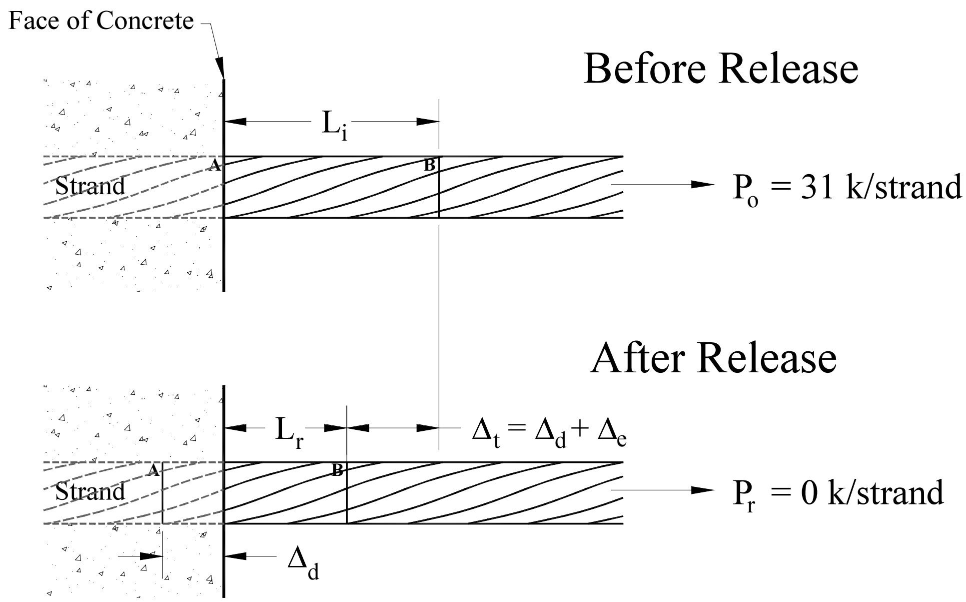

Figure 9.7 Draw-In Illustration

9.4 METHODS TO DETERMINE TRANSFER LENGTH

Many methods exist for determining the transfer length based on different types of measurements. Two methods for analyzing the data of the DEMEC strain measurements are used in this report. Another method is used to determine the transfer length from the strand draw-in values.

9.4.1 95% Average Maximum Strain

The first method used to determine transfer length was the 95% Average Maximum Strain (95% AMS) method [65] shown in Figure 9.8. The first step involves determining the point at which the strain is constant or the 100% strain plateau. This is a subjective determination based on viewing the stain profile. Once the starting point is defined, the average maximum strain (AMS) can be determined by averaging the data after this point.

The next step is to take 95% of the AMS and plot this line against the strain profile. The first intersection of the 95% AMS line and the strain profile defines the transfer length, L_t. The individual strain profiles with the transfer length determined by the 95% AMS Method are all given in Appendix D.

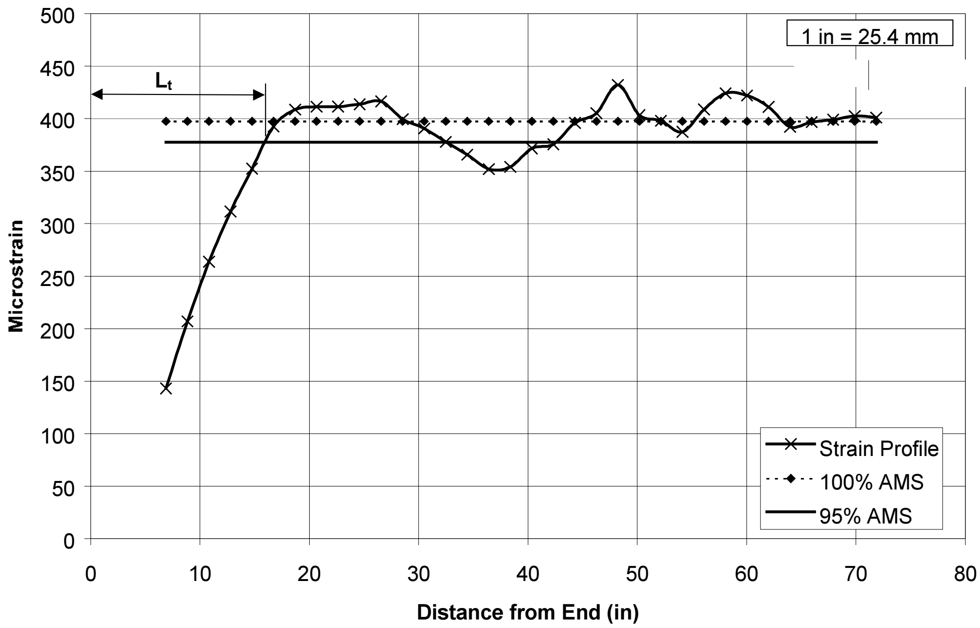

Figure 9.8 95% Transfer Length Method

9.4.2 Slope-Intercept

The other method used with the DEMEC strain data is called the Slope-Intercept Method [54] and is illustrated in Figure 9.9. It is based on the assumption that the strain profile is typically bilinear. It is similar to the 95% AMS Method in that the AMS must be determined. However, a second subjective judgment must be made for this method. It involves determining the slope of the increasing strain portion of the profile. Both the AMS line and the sloped line are then plotted on the strain profile and the intersection of these two lines defines the transfer length, L_t.

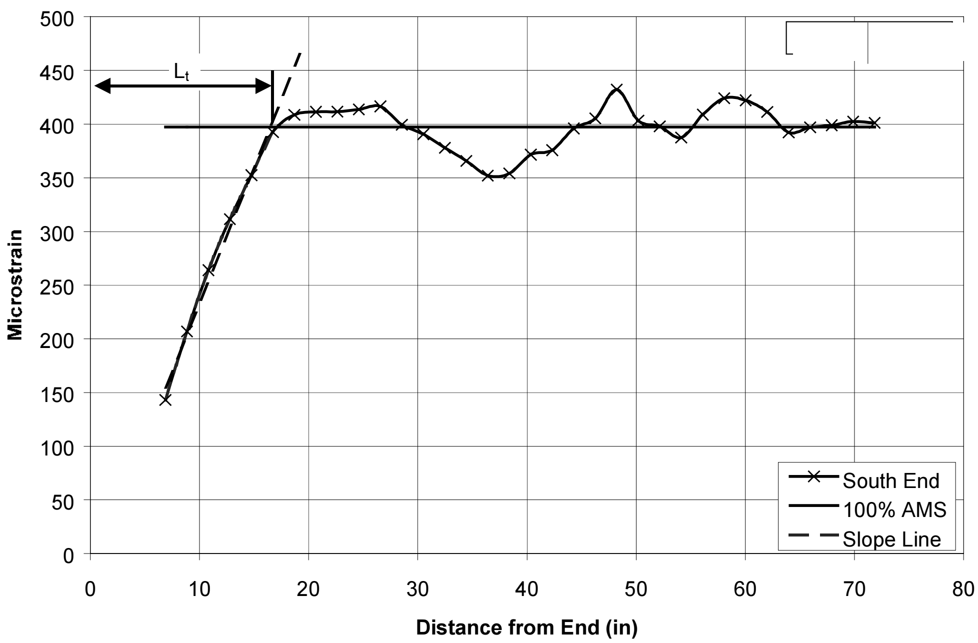

Figure 9.9 Slope-Intercept Method

This method can give very similar results to the 95% AMS Method when the data is bilinear. It was used to verify the results of the 95% AMS Method for some of the beams. It could not be used for all beams because some of the profiles were extremely erratic (more common in the lightweight concrete beams). Use of this method for such profiles led to huge disparities between the value of transfer length from this method and the 95% AMS method. Also, some of the lightweight concrete beam data did not have a bilinear relationship and in those cases correlation between the methods was poor.

9.4.3 Strand Draw-In

The values obtained from measurement of the draw-in can also be used to determine transfer length. The equation used to compute the transfer length from the amount of draw-in is given in Equation 9.3 [30]. In this equation L_t is the transfer length, Δ_d is the measured draw-in, ε_{si} is the initial strain in the steel due to prestress, and α is a factor that adjusts for the type of strain curve. A value of α=2 is used for a bilinear strain profile while α=3 is used for a parabolic strain profile.

$$L_t = \frac{\alpha \cdot \Delta_d}{\varepsilon_{si}}$$ **Equation 9.3**

As discussed in Chapter 7, Russell and Burns [66] use a value of α=2 in their formulation of the transfer length. The notation is different from Equation 9.3, but the equation is essentially the same. The value of α=2 will be used for comparison in this discussion because it is a baseline to which all the values can be compared. The corresponding equation for transfer length is given in Equation 9.4.

$$L_t = \frac{2E_{ps}}{f_{si}} \Delta_d$$ **Equation 9.4**

9.5 BEAM TEST RESULTS

This section gives the results of the DEMEC strain and strand draw-in measurements for determining transfer length on the beam specimens.

9.5.1 DEMEC Strain Measurement Results

The smoothing and averaging of the raw strain profile data was helpful in removing some of the irregularity in the different profiles. Despite this, it was still difficult to determine exactly where the strain plateau began on some of the strain profiles. A best estimate of the beginning of the plateau was found and the transfer lengths determined by the 95% AMS method with the results given in Table 9.1. As mentioned in Section 9.4.2, the slope-intercept method was used to verify the 95% AMS method results on some of the strain profiles, but these results are not given here.

Due to the amount of scatter in the data, it was decided that all the data from the strain profiles of similar beams should be averaged. This gave a much clearer picture of what was occurring in the beams. These averaged strain profiles (ASP) are displayed as the heavy lines in Figure 9.10. The ASP from the normalweight beam data produced a bilinear profile that was expected. The lightweight beams did not produce strain profiles that were typical of those previously reported and found in normalweight concrete beams. The LW6000 and LW8000 beams had strain profiles that were very similar. These profiles are unique among transfer length tests in that they have a preliminary plateau but then rise appreciably again before the strain becomes constant. If this were a random occurrence, the profiles of different beam sets should not correlate so closely.

The ending plateau strain of both lightweight concrete curves is the strain expected to be developed in the beam based on adjusting the difference plateau strain of the normalweight concrete beams in proportion to their different moduli of elasticity. The calculated ratio of the average lightweight concrete AMS to the average normalweight concrete AMS was 1.95. The calculated ratio of the average lightweight concrete elastic modulus to the average normalweight concrete modulus was 1.86. This correlation is evidence that the ASP's were accurate in indicating the strain plateau and therefore a reasonable approximation of the strain profile. Also displayed on Figure 9.10 is a ±10% band. Most of the data fits within this band. Therefore it seems that the averaging of data is meaningful.

Table 9.1 95% AMS Transfer Length Results

Beam ID	Transfer Length, in	Transfer Length, mm
NW6000-1-N	20.9	530
NW6000-1-S	15.7	400
LW6000-20-N	37.2	945
LW6000-20-S	37.6	955
LW6000-1-N	37.8	960
LW6000-1-S	18.3	465
LW6000-2-N	33.9	860
LW6000-2-S	20.7	525
LW8000-20-N	37.2	945
LW8000-20-S	37.0	940
LW8000-1-N	36.6	930
LW8000-1-S	24.8	630
LW8000-2-N	35.4	900
LW8000-2-S	40.7	1035
LW8000-3-N	33.9	860
LW8000-3-S	36.6	930
Ave. NW6000	18.3	465
Ave. LW6000	30.9	785
Ave. LW8000	35.3	896

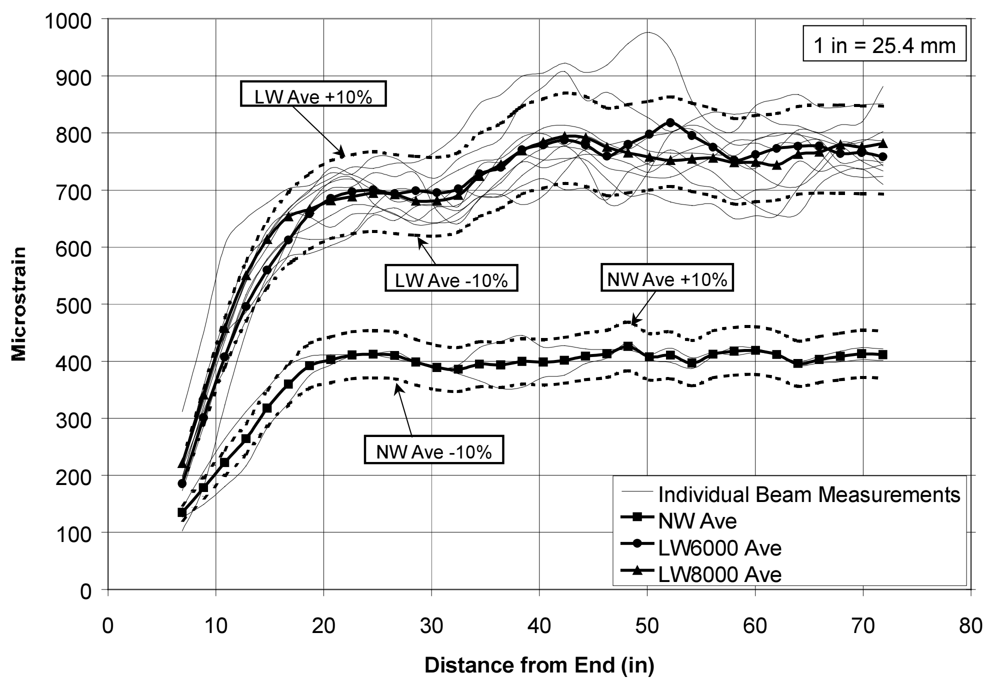

Figure 9.10 Average Strain Profiles

Using the averaged strain profiles, the 95% AMS method was again applied to the data. The results are given in Table 9.2. The good agreement between the two lightweight mixes is expected since both concrete mixes had similar moduli of elasticity. Also, the two concretes did not vary in strength as much as expected. The transfer length for the ½ in. diameter strands is lightweight concrete beams were around 70 diameters which was almost double the 37 diameter transfer length in the normalweight concrete beams.

Table 9.2 Averaged Profile Transfer Lengths

Beam ID	Average Strain Profile Transfer Length, in / (mm)
NW6000	18.2 (463)
LW6000	35.8 (910)
LW8000	34.4 (875)

9.5.2 Draw-In Results

The draw-in values were determined from Equation 9.2 and these values are given in Table 9.3. This table compares the draw-in associated with all the strands, only the bottom strands, and only the top strands along with the maximums and minimums within all these groups (the strand on which these occurred is identified). This was done to see if there was any significance in the amount of draw-in due to the number of strands in an area, which did not seem to be a factor.

Table 9.3 Comparison of Draw-In Values

Beam ID	Strand Data					Notes
	Average All	Max All	Min All	Average Top	Average Bottom	
NW6000-1-N	0.047	0.056 (3)	0.039 (8)	0.047	0.047	
NW6000-1-S	0.054	0.063 (9)	0.041 (1)	0.052	0.054	7,12 bad
LW6000-20-N	0.066	0.100 (9)	0.033 (11)	x	x	zip ties
LW6000-20-S	0.053	0.077 (12)	0.031 (15)	x	x	
LW6000-1-N	0.046	0.066 (7)	0.035 (8)	0.045	0.047	
LW6000-1-S	0.049	0.074 (12)	0.028 (7)	0.047	0.050	
LW6000-2-N	0.047	0.059 (3)	0.032 (2)	0.037	0.049	
LW6000-2-S	0.050	0.062 (1,6)	0.042 (5,7)	0.053	0.050	9-12 bad
LW8000-20-N	0.036	0.064 (10)	0.020 (15)	x	x	zip ties
LW8000-20-S	0.038	0.054 (3)	0.021 (16)	x	x	
LW8000-1-N	0.045	0.057 (1)	0.040 (12)	0.050	0.044	
LW8000-1-S	0.042	0.067 (8)	0.015 (12)	0.041	0.042	
LW8000-2-N	0.039	0.051 (9)	0.034 (4,7)	0.050	0.042	2 bad
LW8000-2-S	0.042	0.054 (9)	0028 (5)	0.045	0.042	
LW8000-3-N	0.044	0.060 (8)	0.027 (2)	0.036	0.046	
LW8000-3-S	0.044	0.061 (2)	0.030 (4)	0.059	0.040	3,7-8 bad
LW6000	**0.052**			**0.045**	**0.049**	
LW6000*	**0.049**					
LW8000	**0.041**			**0.047**	**0.042**	
LW8000*	**0.042**					
NW6000	**0.050**			**0.049**	**0.051**	

Notes:
All units in inches (1 in = 25.4 mm)
** - Indicates that data from zip ties is not used in average*
( ) - Indicates strand number

Despite the significant variance in the maximum and minimum values, it is interesting to note that there was not much variance in the amount of draw-in among all the 6,000-psi mixes. Including the normalweight concrete but not including the zip-tie measurements, the average values ranged between 0.046 in and 0.053 in. The range of values for the 8,000-psi mixes was between 0.038 in and 0.045 in. It is also interesting to note that both the lightweight and normalweight 6,000-psi concrete mixes had average values of 0.049 in and 0.050 in. These beams were released on the same day, which might be a factor because all the measurements were done at the same time. The 8,000-psi mixes were cast several days later and the average for these beams was 0.042 in. Whether this was a function of the strand used in the beams or a correlation between the strengths of the beams is unknown.

The results from applying Equation 9.3 to the draw-in values are given in Table 9.4. The minimum and maximum values that are generally associated with α for normalweight concrete are given. This range will typically encompass the value obtained from the DEMEC measurement results [15,40]. The last column of the table is the α that was obtained from Equation 9.3 when the transfer length from the DEMEC measurement results is substituted into the equation. Values obtained from measurements using the zip-tie method are also included for comparison but these values are not included in the averages to keep the test method consistent. It should be noted that the values for α using the DEMEC data gives values between 2 and 3 for the normalweight concrete. However, most of the values for α in the lightweight beams exceed 3. This indicates that this method may not be valid for lightweight concrete. The shape of the average strain profiles in Figure 9.10 is another indicator that this method may not be accurate. The profiles do not have a bi-linear or parabolic shape, which the equations are based on.

Table 9.4 Transfer Length Values from Draw-In Testing Using Eqn. 9.3

Beam	Average Draw-In	L_t (α=2)	L_t (α=3)	α using Demec
NW6000-1-N	0.0466	14.6	21.8	2.87
NW6000-1-S	0.0535	16.7	25.1	1.88
LW6000-20-N*	0.0660	20.6	30.9	3.61
LW6000-20-S	0.0526	16.4	24.7	4.58
LW6000-1-N	0.0463	14.5	21.7	5.23
LW6000-1-S	0.0493	15.4	23.1	2.38
LW6000-2-N	0.0469	14.7	22.0	4.62
LW6000-2-S	0.0504	15.7	23.6	2.63
LW8000-20-N*	0.0362	11.3	17.0	6.58
LW8000-20-S	0.0378	11.8	17.7	6.27
LW8000-1-N	0.0450	14.1	21.1	5.21
LW8000-1-S	0.0415	13.0	19.5	3.83
LW8000-2-N	0.0387	12.1	18.1	5.86
LW8000-2-S	0.0422	13.2	19.8	6.18
LW8000-3-N	0.0442	13.8	20.7	4.90
LW8000-3-S	0.0443	13.8	20.8	5.29
NW6000	**0.0501**	**15.6**	**23.5**	**2.38**
LW6000	**0.0491**	**15.3**	**23.0**	**3.88**
LW8000	**0.0420**	**13.1**	**19.7**	**5.36**

Notes:
All units in inches (1 in = 25.4 mm)
* - Indicates that zip ties were used
Average values do not include values from * data

9.6 DISCUSSION OF BEAM TEST RESULTS

This section will discuss the results of the beam transfer length testing. Comparisons of the results will be made to concrete properties as well as the equations that attempt to describe transfer length.

9.6.1 Comparison of Methods

The average values found for the transfer length from all the methods are given in Table 9.5. This gives a concise example of the variability that is inherent in determining transfer length. The variability in the lightweight concrete data is much greater than that of the normalweight concrete. This is due to the fact that most of the methods to determine transfer length were developed for normalweight concrete.

Table 9.5 Comparison of Transfer Length Methods

Beam ID	Individual Strain Profiles, in / (mm)	Average Strain Profiles, in / (mm)	Draw-In (α=2) in / (mm)	Draw-In (α=3) in / (mm)
NW6000	18.3 (465)	18.2 (463)	15.6 (397)	23.5 (596)
LW6000	22.1 (562)	35.8 (910)	15.4 (390)	23.0 (585)
LW8000	29.7 (755)	34.4 (875)	13.1 (333)	19.6 (499)

The table indicates that the draw-in results are completely different than the DEMEC data results. Based on draw-in data and equations, the LW8000 beams have the smallest transfer lengths. This is directly opposite of the data from the strain profiles which are a more direct measure of the transfer length. The average values from the individual strain profiles seem to indicate that the transfer length increases with the strength of the concrete. However, this is startling because conventional equations usually equate a higher concrete strength with smaller transfer lengths. The trend of smaller transer lengths is also shown by the strand draw-in data. The equation for draw-in have never been verified as applicable to lightweight concrete. The averaged strain profile values give more fundamental data that seems credible since their transfer lengths can be clearly identified from the strain profiles with less error of interpretation. The fact that the transfer lengths for the lightweight concrete mixes are similar for this method is reasonable because the strengths of these mixes did not vary by much and they both had similar moduli of elasticity.

As mentioned in Chapter 8, only six full size beams were cast along with two preliminary 20-ft (6.1-m) beams. This gave a total of 2, 6, and 8 sets of data for the NW6000, LW6000, and LW8000 beams for determining transfer length, respectively. Therefore, the following discussion must be viewed as preliminary due to the small sample size. Also, it should be noted that two different materials are being compared while the models for determining transfer length were developed for normalweight concrete and not lightweight concrete.

9.6.2 Comparison to Concrete Properties

This section compares the transfer length and the concrete properties. The properties to be compared are the strength of the concrete and the modulus of elasticity.

Some of the equations used to define transfer length empirically use the strength of the concrete as a term in the equation [83,54]. Therefore, whether a trend existed between the concrete compressive strength and transfer length was examined. A comparison of the concrete strength and ASP transfer length values are given Table 9.6. Figure 9.11 displays these values along with the individual strain profile values. The graph seems to indicate that as the concrete strength increases, so does the transfer length. It is difficult to conclude that this is the actual trend of the data since any number of different sloping lines could be drawn through the lightweight concrete data points. Viewing the normalweight and lightweight concrete beams separately, the lightweight data has a similar transfer length despite the different strengths of the concrete. No clear correlation is apparent from the data. Previous researchers have found trends related to concrete compressive strength [83,54]. This is possible when the same type of material is used between tests. Applying the trend of previous research to the data in this study, as the concrete compressive strength increases, the transfer length should decrease. As stated previously, this is not the case for the data presented.

Table 9.6 Comparison of Transfer Length and f'_{ci}

Beam ID	f'_{ci} psi	Ave. Strain Profile Transfer Length, in
NW6000	3,849	18.2
LW6000	4,902	35.8
LW8000	5,563	34.4

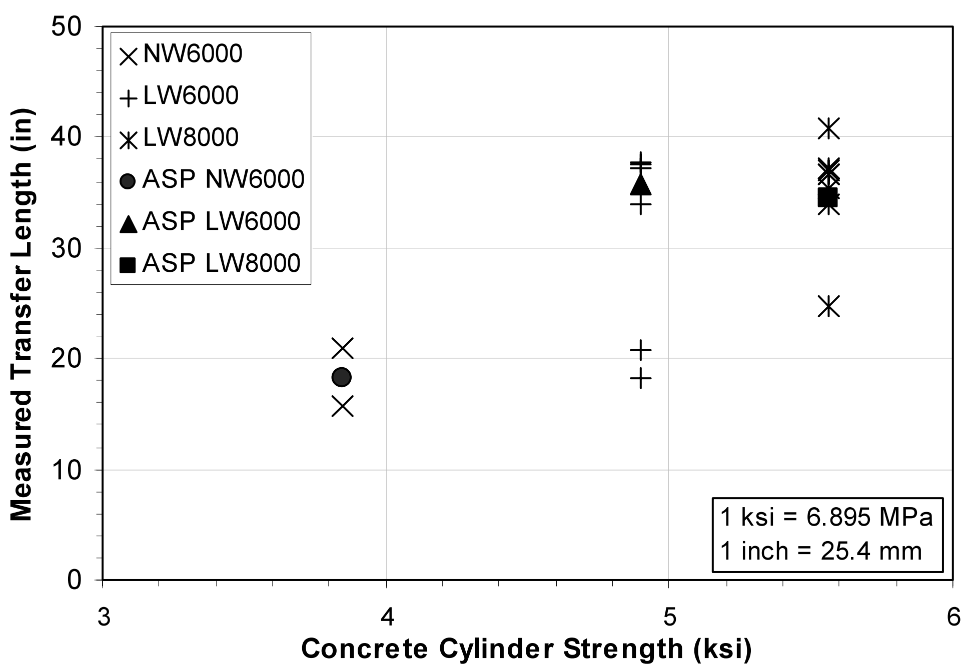

Figure 9.11 Comparison to Concrete Strength

Some equations that describe transfer length use the square root of f'_c as a variable. The ACI 318 Code gives an equation that relates the modulus of elasticity of normalweight concrete to the square root of f'_c [54]. The conclusion that can be drawn from this is that the modulus of elasticity may affect transfer length. It is a logical assumption that the modulus of elasticity should affect the transfer length because it is a measure of the stiffness of the concrete and therefore affects the strains in the concrete, which determines the transfer length. To examine if a correlation between these properties existed in the data for this study a plot of the modulus of elasticity versus the transfer lengths was created. This plot is shown in Figure 9.12.

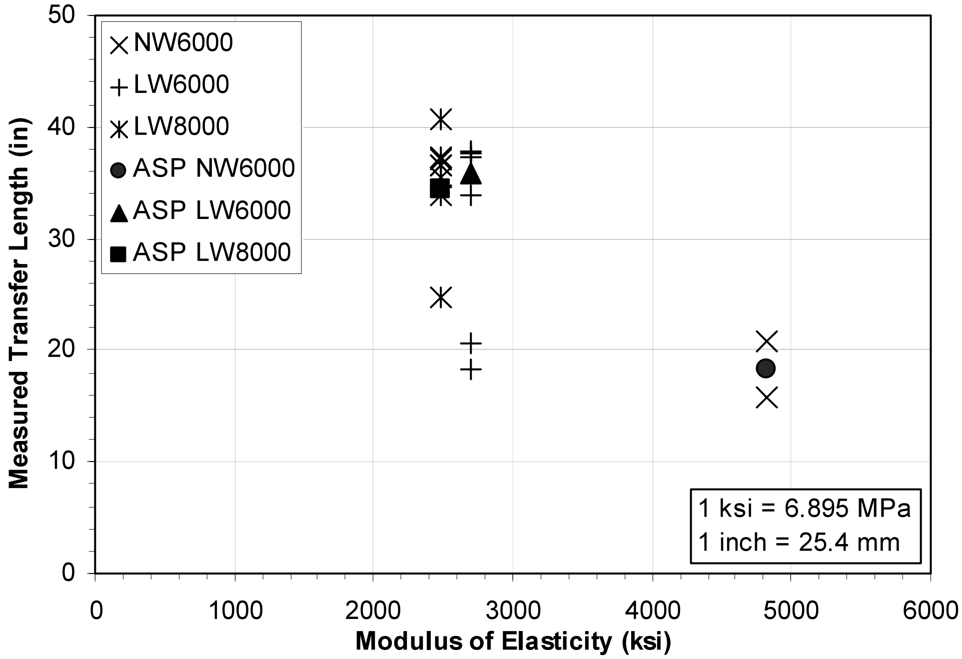

Figure 9.12 Modulus of Elasticity vs. Transfer Length

The trend from the plot indicates that as the modulus of elasticity increases, the transfer length decreases. This is consistent with an equation developed by Buckner for determining transfer length, which uses the modulus in the denominator [40]. This model will be examined further in Section 9.6.3.

Table 9.7 gives a correlation between the transfer length and initial modulus of elasticity, E_{ci}. Included in this table is the transfer length divided by the inverse of the modulus. If the modulus of the concrete was a factor that influenced the transfer length, then these values should be similar. These values are similar, which supports the theory that modulus plays an important role in determining transfer length.

Table 9.7 Comparison of Transfer Length and Modulus of Elasticity

Beam ID	E_{ci} ksi	Ave. Strain Profile Transfer Length, in	$\frac{L_t}{(1/E_{ci})}\times 10^{-3}$
NW6000	4,829	18.2	88.0
LW6000	2,693	35.8	96.5
LW8000	2,489	34.4	85.7

9.6.3 Comparison to Transfer Length Equations

There are many equations that attempt to describe transfer length. Most of these equations are based on studies of normalweight concrete and therefore may not apply to lightweight concrete. This section will compare the transfer lengths with some of the popular expressions for transfer length. More importantly, it will compare the transfer length to the ACI Building Code [3] and AASHTO Code [1] equations, which are used for design.

Each comparison uses a graph that plots the predicted transfer length versus the measured transfer length. The dashed line on each graph represents complete agreement between the predicted and measured values. For the model that a graph represents to be judged as quite conservative, a preponderance of the data should fall below the dashed line.

Table 9.8 gives the values for the calculated transfer lengths for the different beams. These expressions were chosen because of the difference between the expressions. Some depend on the diameter of the strand alone and others depend on the concrete strength or the modulus of elasticity. From the previous discussion of transfer length in comparison to concrete properties, it is probable that equations using the modulus of elasticity as a variable will give better correlation with the results.

Table 9.8 Transfer Length Equations and Predicted Values for Beams

Author	Transfer Length Equation	L_t NW6000	L_t LW6000	L_t LW8000
ACI 318[2]	$L_t = \frac{f_{se}}{3} d_b$	32.1	30.7	30.5
	$L_t \approx 50 \cdot d_b$	25.0	25.0	25.0
AASHTO Shear Provisions [1]	$L_t = 60 \cdot d_b$	30.0	30.0	30.0
Russell & Burns [39]	$L_t = \frac{f_{se}}{2} d_b$	48.2	46.1	45.7
Zia & Mostafa [46]	$L_t = 1.5 \frac{f_{si}}{f'_{ci}} d_b - 4.6$	35.0	26.5	22.8
Buckner (FHWA) [13]	$L_t = \frac{1250 f_{si} d_b}{E_c}$	26.3	47.1	51.0
ASP*	Measured	18.2	35.8	34.4

* Averaged strain profiles

In some studies of normalweight concrete it was found that the ACI 318 Code equation was a conservative estimate of the transfer length [15]. This is also the case for the normalweight concrete beams measured in this study, as shown in Figure 9.13. This figure compares the ACI 318 Code equation using f_{se} as a variable in the equation for transfer length (Eqn. 2.14). The value for f_{se} was calculated based on the recommendations of ACI 318 [82]. All the normalweight data falls below the line that signifies complete agreement between equation and measurement. This equation is not conservative for most of the lightweight concrete data, except for three points. This equation was formulated to be a conservative estimate of the transfer length. Therefore, it is reasonable to conclude that the ACI 318 Code equation does not model the transfer length for lightweight concrete based on the limited data sample from this study.

The ACI 318 Code gives an approximation to the transfer length equation as $50d_b$ [3]. This approximation is based on an effective stress in the strands of 150 ksi (1,034 MPa). This stress is usually reached at some time after 28 days. The beams in this study were measured the day of release and therefore had an effective stress above 180 ksi (1,241 MPa) for all beams. It is clear from Figure 9.14 that this model only fits the normalweight concrete beam. Therefore, it is conservative in the case of normalweight concrete because the effective stress in the strand is much higher than the model assumes. It also indicates that the model is less conservative than the ACI 318 equation for transfer length in this study because the effective stress used in the approximation is lower than the actual stress, leading to smaller transfer lengths.

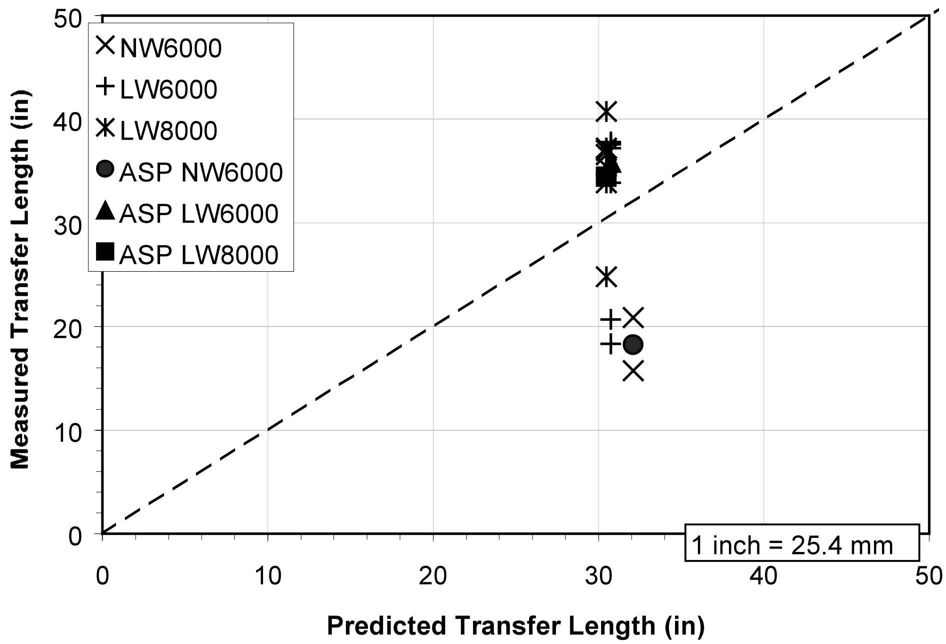

Figure 9.13 Comparison to ACI 318 Code

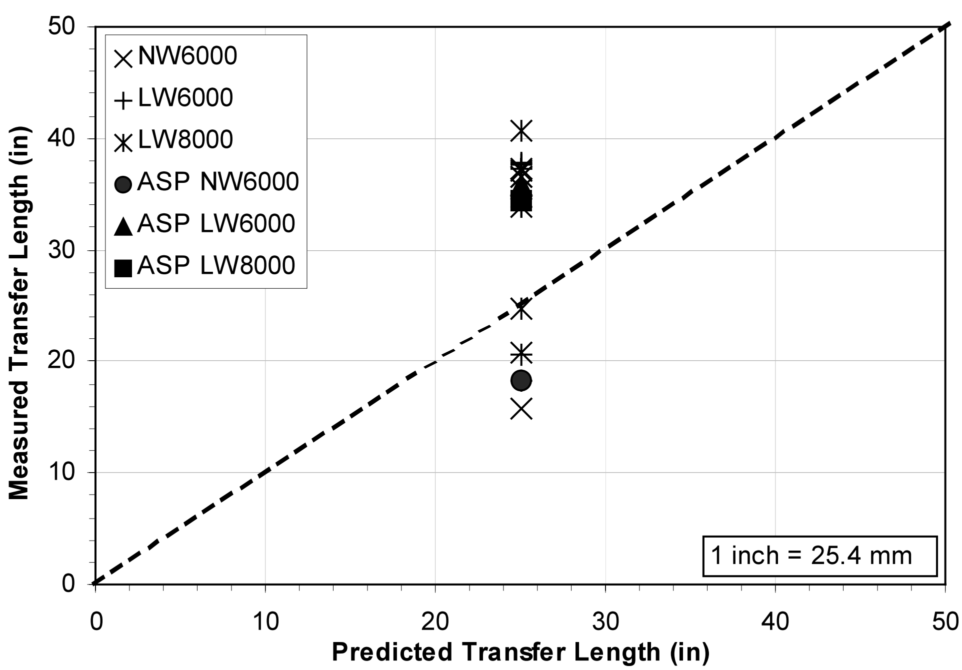

Figure 9.14 Comparison to ACI 318 Approximation

The data for the lightweight concrete beams is nearly 10 in (254 mm) on average above the predicted value for transfer length. Only two data points from all the lightweight data falls below the line that indicates agreement between model and measured data. Since this model is dependent on f_{se} equal to 150 ksi (1,034 MPa) and was developed for normalweight concrete it cannot be used to accurately determine transfer length in lightweight concrete beams.

The new AASHTO Shear Specifications are more conservative than the ACI 318 Specifications and consider the transfer length to be $60d_b$ [1].This is done to update the code to the current practices of prestressing where higher initial prestresses are used and low relaxation strands are common. Despite this, as shown in Figure 9.15 the model still underestimates the transfer length in lightweight concrete and a similar argument can be given as the one stated above for the ACI 318 approximation. The data for the normalweight concrete is again conservative, as would be expected due the increased multiplier on the strand diameter.

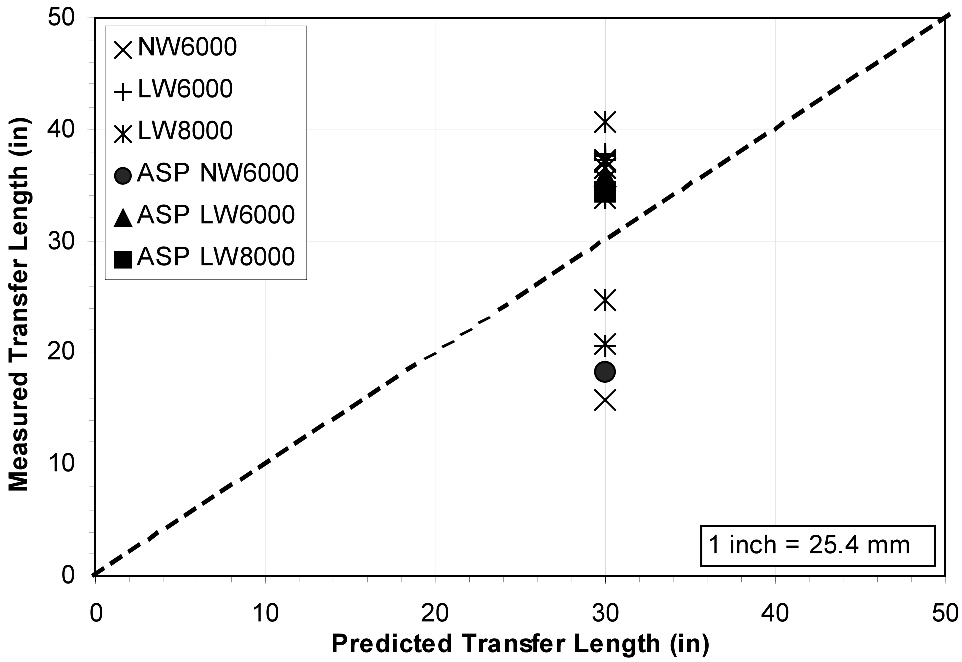

Figure 9.15 Comparison to AASHTO Shear Provisions

Russell and Burns developed a model that included much of the existing data on transfer length at that time, not just the data from one study. The result was an equation very similar to ACI 318, but with a different stress in the denominator [66]. Figure 9.16 shows that Russell and Burns's model covers all the data including the lightweight values and is conservative. It includes all the values from the lightweight beams with a factor of safety, but overestimates the transfer length in the normalweight beams by over 100%. Despite the fact that this model accurately bounds all the data in this study, it is important to note that the model does not accurately model the trend in the data.

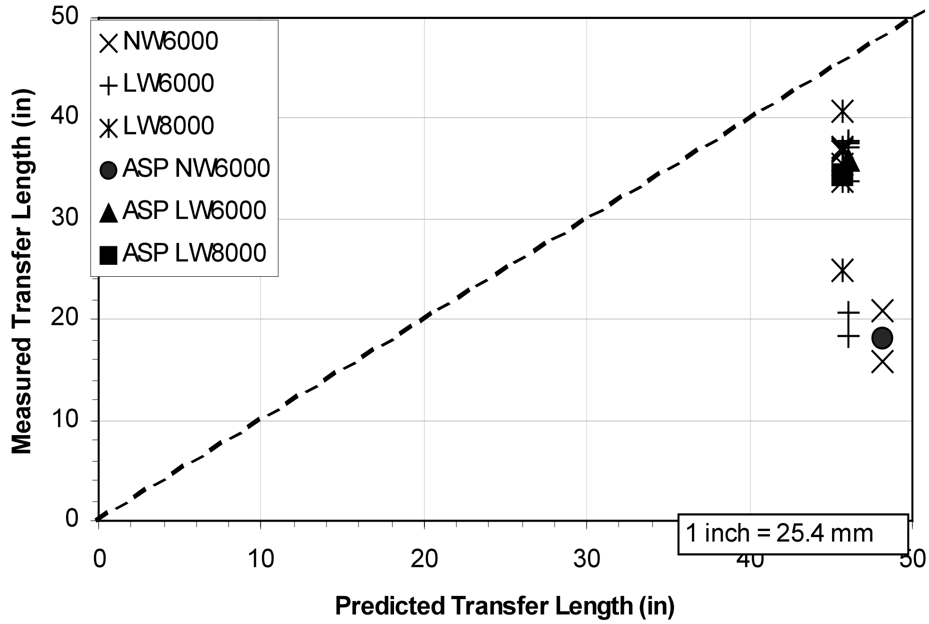

Figure 9.16 Comparison to Russell & Burns

The previous four models used f_{se} and d_b as variables in the equations to describe the transfer length. The only model that made a conservative estimate for the transfer length was the Russell and Burns model. All the models unanimously failed to predict the correct trend of the data. Therefore, using only the strand diameter to predict transfer length for normalweight and lightweight concrete is not possible. The two equations that used f_{se} as a variable predicted a trend that was opposite that of the data. This indicates that these variables alone cannot describe the behavior of normalweight and lightweight concrete transfer lengths together.

The next two models use a combination of the two variables described above, f_{se} and d_b. Also included in these models are properties of the concrete. The Zia and Mostafa model uses concrete compressive strength as a variable and the Buckner model uses the modulus of elasticity of the concrete as a variable. As shown earlier, the concrete compressive strength does not seem to affect the transfer length in a way that is expected from past research [54,77]. Specifically, as concrete strength increases, the transfer length should decrease.

In the Zia and Mostafa model the concrete compressive strength is in the denominator (Table 9.8). Therefore, since the compressive strengths of the lightweight concretes in this study were larger than the normalweight concrete, the model would predict smaller transfer lengths for the lightweight concrete beams. However, as discussed earlier, the trend between strain profiles for normalweight and lightweight concrete was not typical. Therefore, despite the increased strength of the lightweight concrete over the normalweight concrete, it is not expected that this model would fit the lightweight data well. This is exactly what the data shows, as shown in Figure 9.17. Again, as was seen in the comparison with only the concrete strength, the trend of the data is opposite to the predicted line.

The reason for the opposite trend in the data when the compressive strengths are compared to the transfer lengths is due to the difference in elastic modulus between the normalweight and lightweight concretes. A model that accounts for this difference was developed by Buckner [21]. It uses the modulus of elasticity as a variable in the denominator of the transfer length (Table 9.8) and therefore fits the relationship discussed between the transfer length and elastic modulus discussed earlier. Figure 9.18 shows that the model is conservative for all the data and follows the trend of the data. Also, since the modulus of the lightweight concrete tends to level out despite increases in compressive strength [33], one would expect to see the lightweight data gather in the same area. The data seems to follow this trend, but more testing needs to be performed before a definitive trend can be established.

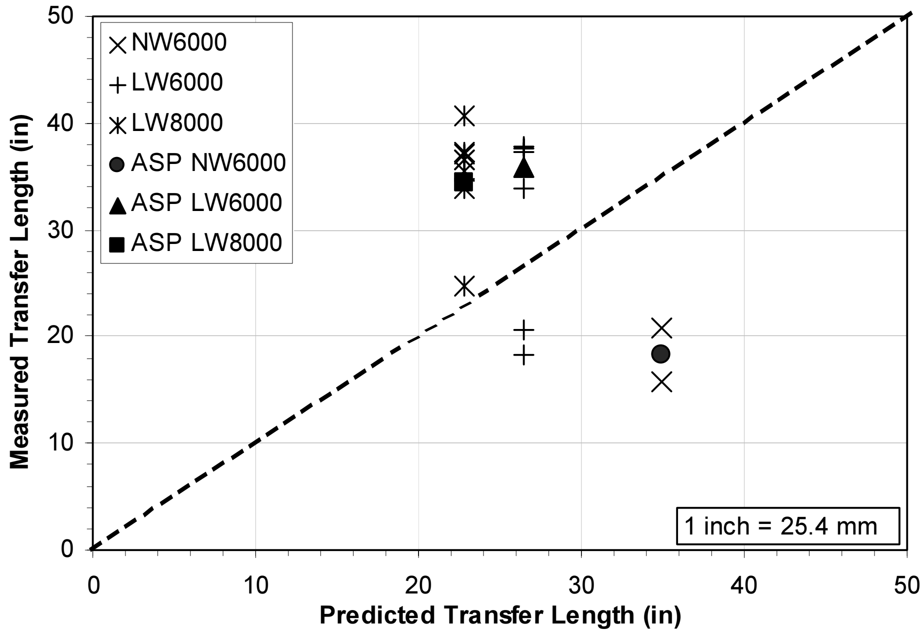

Figure 9.17 Comparison to Zia & Mostafa

The figure also shows a similar amount of conservatism between data sets that was not seen in the Russell and Burns model. Changing the constant in this equation could lead to an accurate representation of the transfer length. This is shown in the figure by the bold line that best fits the averaged strain profile data. The results of this comparison leads to the conclusion that an accurate model that incorporates lightweight as well as normalweight concrete can be developed for transfer length. The key to the accuracy of this model is the use of the modulus of elasticity in the equation that describes behavior, with the results shown in Figure 9.18.

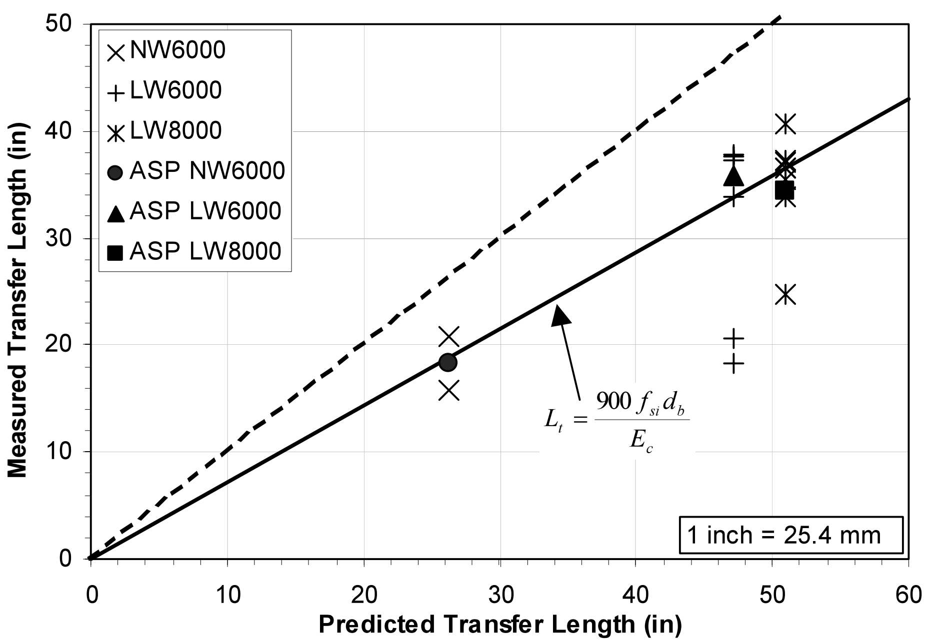

Figure 9.18 Comparison to Buckner

The last comparison made is between the draw-in data and the measured transfer lengths. The dashed line in Figure 9.19 represents Equation 9.4 [66]. This graph shows that this model tends to underestimate the transfer length for all the data. This may be because the draw-in is more affected by the concrete strength than the elastic modulus. This is definitely the trend exhibited by the data. The concrete strengths of the NW6000 and LW6000 mixes were similar at the time of draw-in testing and the average of the measured values was very close. The strength of the LW8000 concrete mix was higher and the average of the draw-in values was less than those of the other mixes.

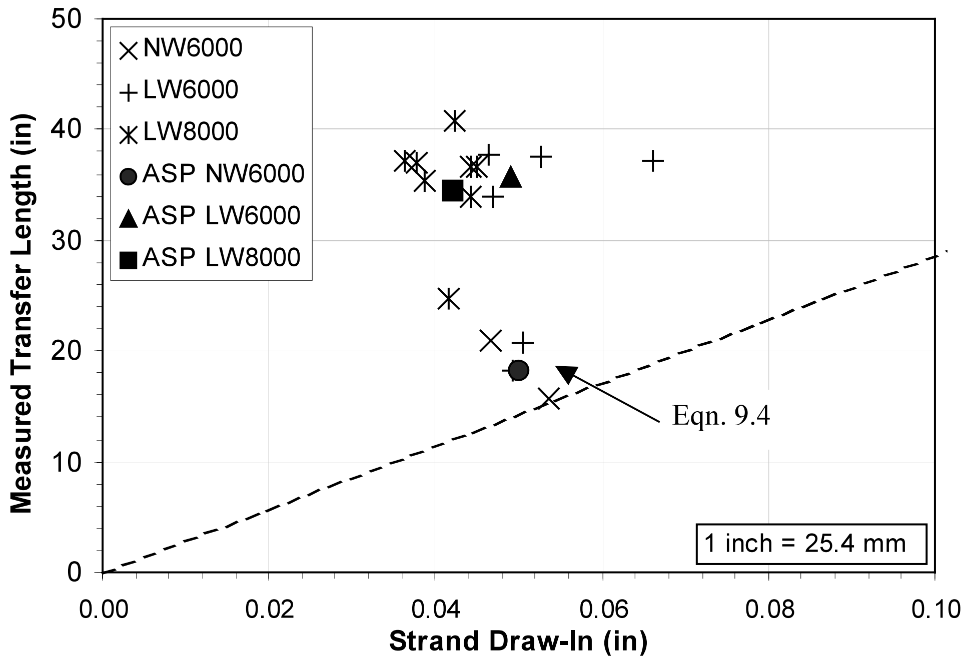

Figure 9.19 Comparison of Strand Draw-In to Transfer Length

9.7 TRANSFER LENGTH IN LIGHTWEIGHT CONCRETE DECK PANELS [85]

Because the lightweight concrete beam measurement indicated longer than expected transfer lengths, it was decided to determine the transfer length of 3/8-inch pretensioning strands used in precast prestressed lightweight concrete panels. A test series was run by G. Sylva and is reported in Reference 85. This series is the basis for Section 9.7. As shown in Figure 9.20, these prestressed concrete panels span between bridge girders and become permanent members of the composite deck. Use of these panels eliminates a considerable portion of the formwork required for constructing the deck. Also, the panels provide an instantaneous surface that can be used for constructing the cast-in-place portion of the deck.

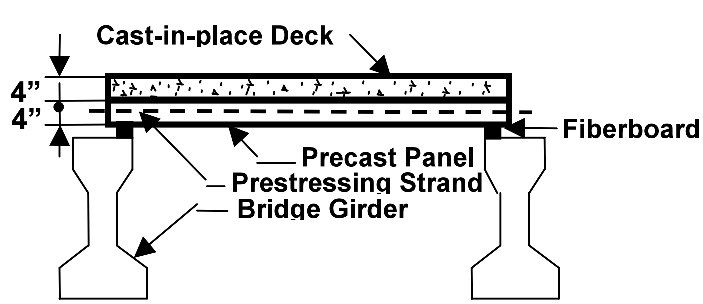

Figure 9.20 Precast Panel Stay-in-Place Forms

9.7.1 Transfer Length Test Setup

Figure 9.21 depicts the overall dimensions of the prestressed concrete panels and placement of the DEMEC points that were used for the strain measurements. A total of six (6) panels were cast using either a plant mix normalweight concrete or a lightweight concrete obtained from a local ready-mix supplier.

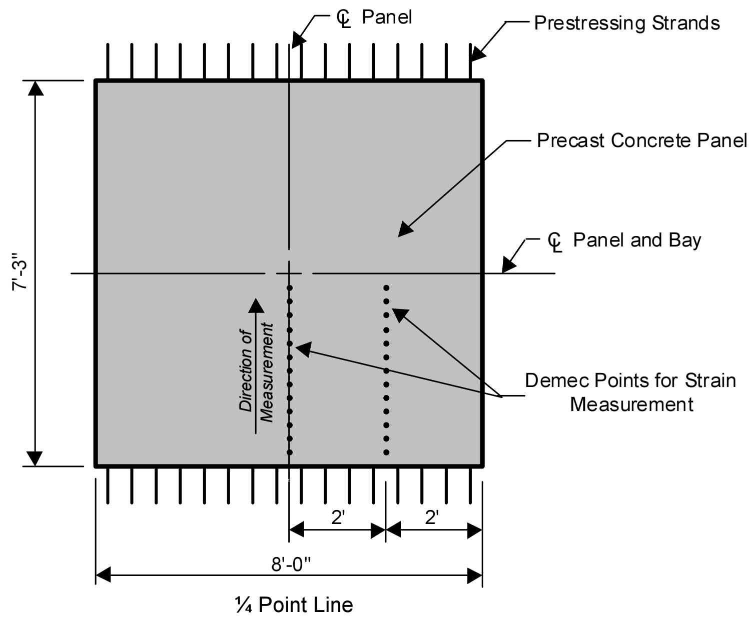

Figure 9.21 Panel Transfer Length Test Specimens

After curing for approximately one day, the panels were instrumented with DEMEC points as shown in the previous figure and as noted in Table 9.9.

Table 9.9 Instrumentation Locations for Panel Transfer Length Tests

Panel ID	Concrete Type	Demec Pts. At CL	DEMEC Pts. at ¼ Pt.
D52	Normalweight	✓	
D53	Normalweight	✓	✓
D54	Normalweight	✓	✓
D55	Lightweight	✓	
D56	Lightweight	✓	✓
D57	Lightweight	✓	✓

9.7.2 Test Procedure

The strain measurements were taken before release, after release, and then 85 days later. Measurements were performed using a digital DEMEC device and multiple readings were taken at each point until duplicate readings were obtained. The duplicate readings were done to minimize errors in the data. After the data was recorded, the points along each line were smoothed using the same method described by Kolozs [43] and used for previous work done on this project. In general, the smoothing of the points involved taking three consecutive point readings and applying their average to the middle point. Additional smoothing was then performed by averaging the points along the centerline with the corresponding point along the ¼ point line. This was done for panels D53, D54, D56, and D57.

9.7.3 Test Results

Figures 9.22 and 9.23 show the strains for the panels. Figure 9.22 contains the smoothed and averaged strain measurements immediately after release of the prestressing strands, while Figure 9.23 contains the strain measurements 85 days after release.

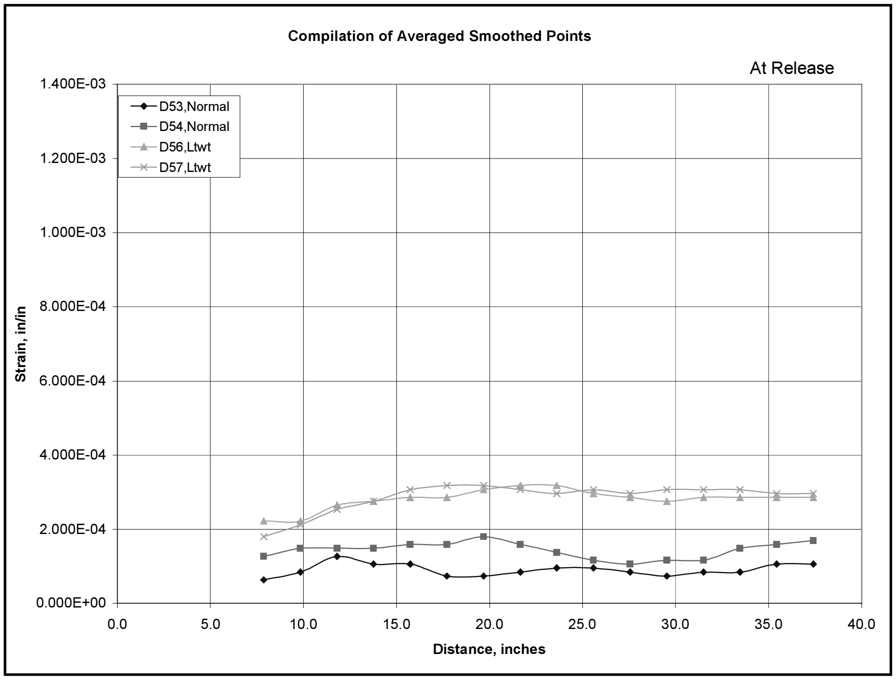

Figure 9.22 Transfer Length Panel Initial Strain Measurements

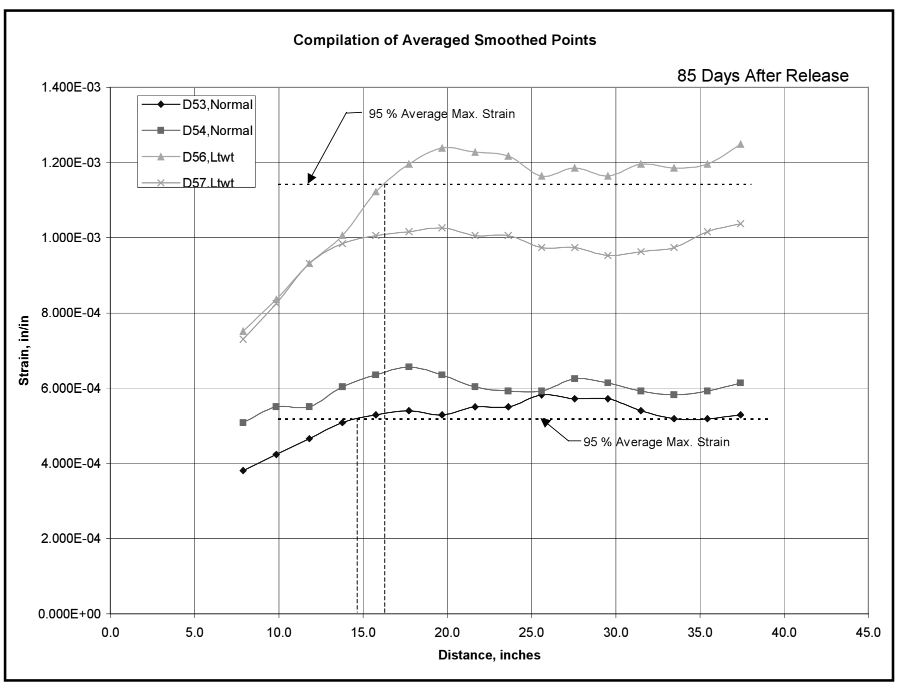

Figure 9.23 Transfer Length Panel Strain Measurements at 85 Days

Both figures show that the strains with the lightweight concrete are approximately twice as large as the strains with the normalweight concrete as might be expected from differing moduli. Using a 95 percent average maximum strain [1] for panels made from both types of concrete, the transfer length for the panels made from the lightweight concrete (43 d_s) is only slightly larger than the transfer length for the normalweight panels (39 d_s). The difference is less than 15 percent. The transfer lengths determined for each of the panel types are also less than the 18.75 inches that would be predicted using the AASHTO Section 9.20.2.4 criteria of 50 times the strand diameter.

9.7.4 Conclusion of the Panel Transfer Length Tests

The purpose of this investigation was to determine the transfer length of 3/8-inch pretensioning strands used in prestressed concrete panels cast from both normal and lightweight concrete. From the data obtained in this investigation, it is evident that the transfer length for 3/8" strands measured in this test for both normalweight and lightweight concretes is less than that predicted using AASHTO transfer length criteria. Further, the transfer length in panels made from lightweight concrete is only slightly (10%) more than in panels made from normalweight concrete. The same AASHTO Section 9.20.2.4 design rules and procedures for transfer length can be used in both type panels.

CHAPTER 10: DEVELOPMENT LENGTH TESTING

10.1 INTRODUCTION

This chapter summarizes the testing of the development length beams. The testing program included five lightweight concrete pretensioned beams and one normalweight concrete beam used as a control specimen. Each test is designated as shown in Figure 10.1.

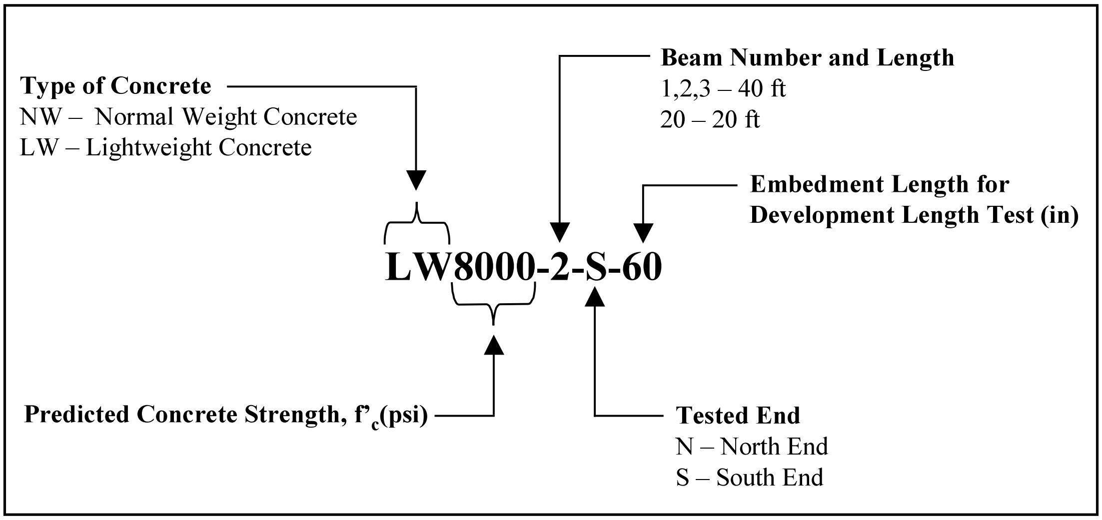

Figure 10.1 Beam Nomenclature

The first two letters indicate whether the beam is composed of normalweight (NW) or lightweight (LW) concrete. The next number indicates the specified design strength of the concrete beam, 6000 or 8000 psi. The number following the dash indicates the particular beam of that strength. Next, the end of the beam tested is indicated by N (for north) or S (for south). The ends of the beam were labeled in the casting yard and kept consistent throughout the testing process. The final number indicates the embedment length (i.e. the distance between the load point and the support) tested on that end of the specimen. For example, the nomenclature listed in Figure 10.1 indicates that the test was performed with an embedment length of 60 in on the south end of the second lightweight beam with specified design strength of 8000 psi.

The full nomenclature will not be used at all times throughout this report. When a particular beam is referenced, with both ends included, the nomenclature will leave off the development length and beam end indicators. Additionally, if a group of beams is referred to, only the first term will be presented (e.g. LW8000 to refer to all of the 8000 psi lightweight beams).

10.2 TEST SPECIMENS

The complete testing program involved a total of twelve tests on six beams, with each beam undergoing two separate tests. Kolozs tested the first three beams and reported the results [43]. Thatcher tested the last three beams and reported the results [75]. The beams were all cast at Heldenfels Prestressing Plant in San Marcos, Texas. They were cast in groups of three over six days, with strand release occurring the day after casting. All cross-section details were the same for each beam; the only variable consisted of the concrete mix specified. One normalweight beam and two lightweight beams were cast the first day, all with a 28-day specified compressive strength of 6000 psi. Two days after the strand release, three lightweight beams with a 28-day specified compressive strength of 8000 psi were cast. Following instrumentation and testing of the transfer length of each of the beams, they were transported to Ferguson Structural Engineering Laboratory (FSEL) at the J. J. Pickle Research Campus of The University of Texas at Austin. Details of the mix designs and the casting of the beams were summarized in Chapters 4 and 5.

10.2.1 Concrete Mix

The concrete mixes used were reported in detail in Chapters 4 and 5. Three mixes were developed: a normalweight mix with a 28-day nominal compressive strength of 6000 psi, a lightweight mix with a 28-day nominal compressive strength of 6000 psi, and a lightweight mix with a 28-day nominal compressive strength of 8000 psi which for actual use should be referred to as a 7500 psi mix. The resulting properties for each are shown in Table 10.1.

Table 10.1 Concrete Properties

Beam ID	Compressive Strength (f'_c), psi		Unit Weight lb/ft^3
	1-day	Long Term	
NW6000	3,490	5,500	149
LW6000	4,900	8,130	118
LW8000	5,560	7,850	122

The long-term strength of the normalweight mix was a bit short of the specified strength. The long-term strength of the lightweight 6000 psi mix was around 8000 psi, significantly higher than the specified strength. The long-term strength of the 8000-psi mix was actually slightly less than the specified strength.

10.2.1.1 Normalweight Aggregate

The aggregate used in the normalweight mix was comprised of ¾ in crushed limestone aggregate readily available in the Austin, Texas region. It resulted in an equilibrium concrete weight at 28 days of approximately 145 lb/ft^3.

10.2.1.2 Lightweight Aggregate

The aggregate used in the lightweight mix was comprised of a ¾ in. Lightweight coarse aggregate called Clodine, obtained from Texas Industries (TxI). Clodine aggregate is an expanded clay. It resulted in an equilibrium concrete weight at 28 days of approximately 118 to 122 lb/ft^3 [33].

10.2.1.3 Other Components

The concrete mixes consisted of water, sand, cement, fly ash, retarder, and superplasticizer, in addition to the aggregate. The amounts of each component for the three mixes are shown in Table 10.2.

Table 10.2 Mix Designs

Material	NW6000	LW6000	LW8000
Water	250	222	247
Cement (Type III)	517	504	671
Fly Ash	0	168	316
¾ in Limestone Coarse Aggregate	1869	0	0
¾ in Lightweight Coarse Aggregate	0	1264	1123
Sand	1355	1149	1123
Retarder	12	12	12
Superplasticizer	20.4	34	54

Notes:
1) Quantities in lbs and oz
2) Quantities per yd^3

10.2.2 Cross Section Details

The cross sections of all of the test beams were the same, with an area of 276 in^2. The beams were I-shaped sections 28 in deep, with a larger bottom flange region than top flange region. Dimensions are shown in Figure 10.2 (Type I AASHTO cross section). The prestressing strands and pattern were the same in each test beam, as were the reinforcing details (see Sections 10.2.3 and 10.2.4).

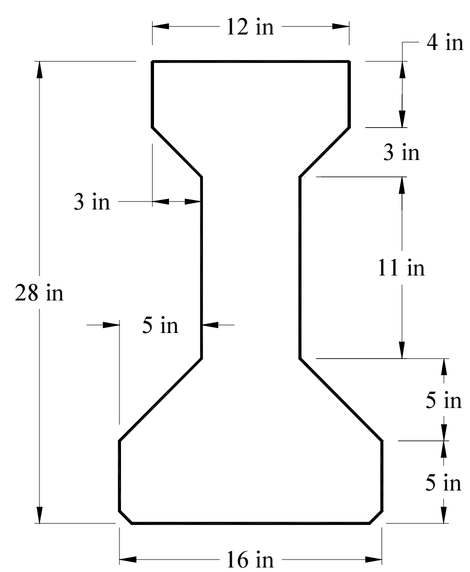

Figure 10.2 Beam Dimensions

10.2.3 Prestressing Strands

The prestressing strands used in the test beams were ½ in diameter, low relaxation, ASTM A416, Grade 270 ksi 7-wire strands. The ultimate stress was reported on the mill certificates as 270 ksi, with an elastic modulus of 28,000 ksi. No independent verification of these values was made. The strands were supplied by American Spring Wire Corporation.

Twelve strands were placed in each test beam in a 2 in grid pattern. Two strands were located in the top flange to ensure compression in that region in the beam ends, and ten in the bottom flange. The bottom flange strands were placed in two rows, to allow a larger moment arm to develop. The strand layout is shown in Figure 10.3.

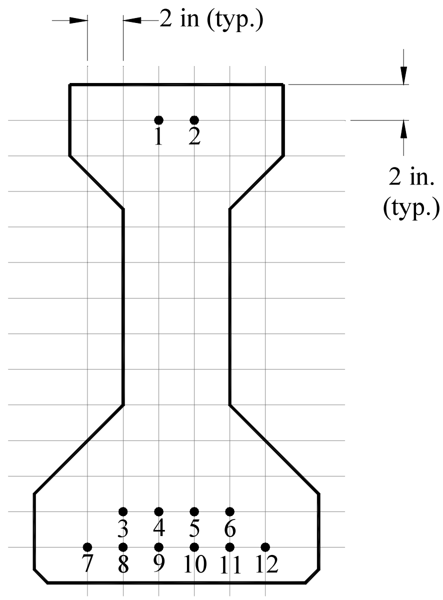

Figure 10.3 Strand Layout

10.2.4 Reinforcement

The non-prestressed reinforcement used in the test beams was Grade 60 steel, and was supplied by Border Steel. The yield stress was reported on the mill certificates as 66 ksi, with an ultimate stress of 100 ksi. No independent verification of these values was made.

The flexural reinforcement was placed in patterns used in similar tests and satisfactory to TxDOT requirements. The shear reinforcement placed in the beams exceeded shear reinforcement typical for similarly sized beams, in order to ensure that the beams would not undergo sudden shear failure during testing due to the short shear spans of the tests. Shear failure of the beams would not have allowed effective determination of the development lengths; thus a flexural failure was ensured by the amount of shear reinforcement used. The layout of the reinforcement is shown in Figures 10.4 and 10.5.

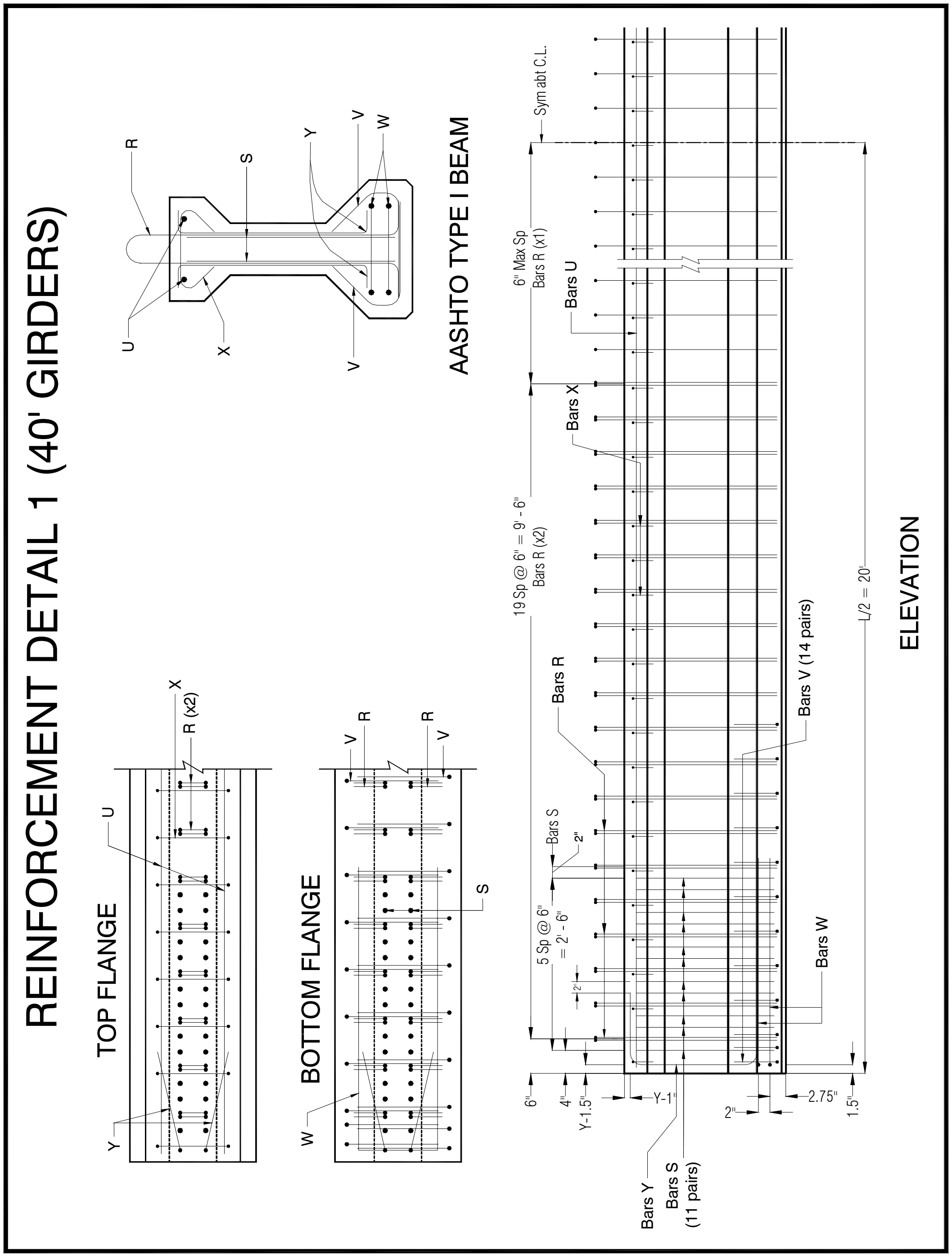

Figure 10.4 Test Beam Reinforcement Details

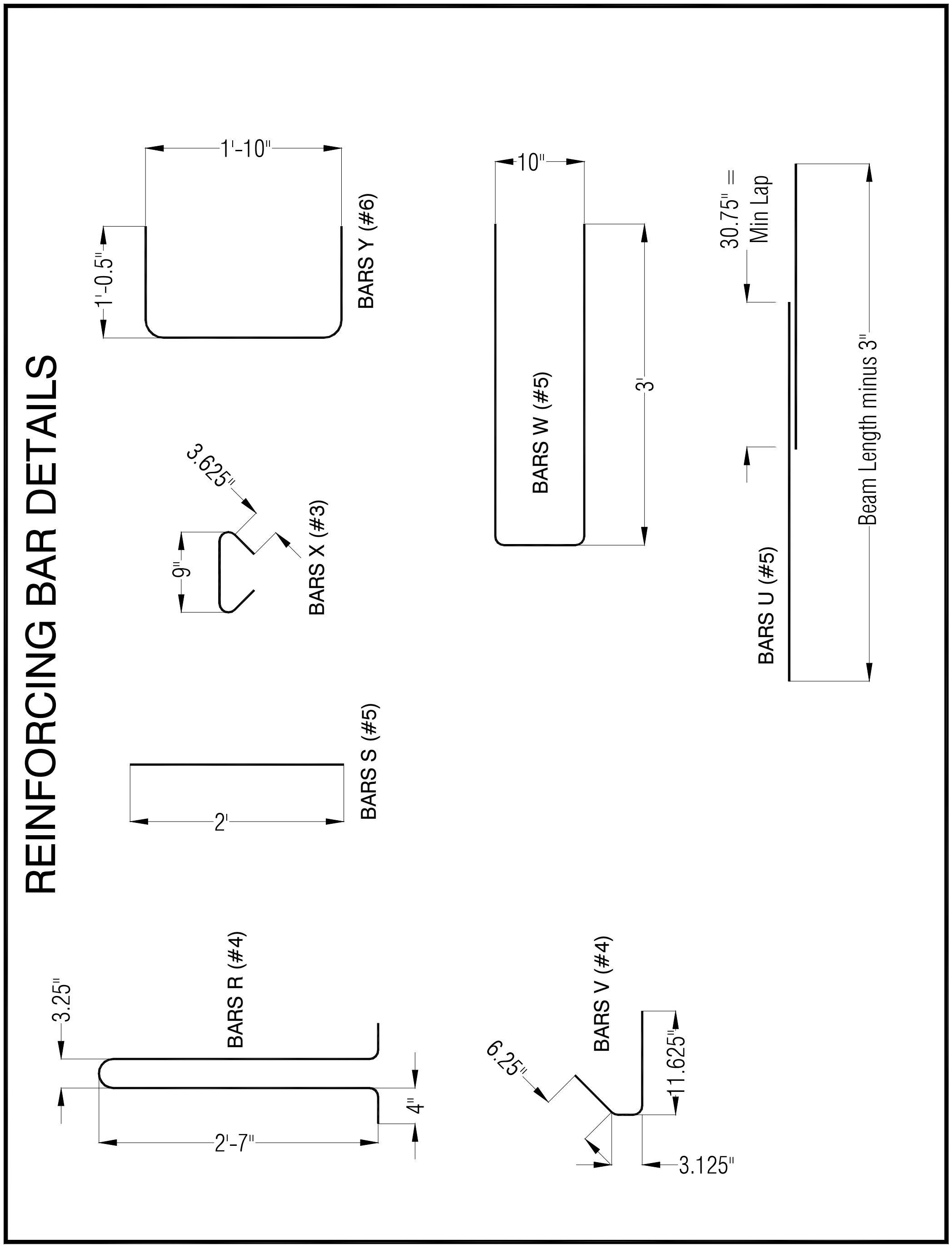

Figure 10.5 Reinforcement Bar Details

10.2.5 Deck & Panels

Each test beam was tested as a composite section that was constructed by placing a concrete slab, or deck, on top of the beam after it was brought into FSEL. The deck components were varied from test to test, but the dimensions were held constant. The reinforcement was designed using AASHTO specifications [1], and is shown in Figure 3.6.

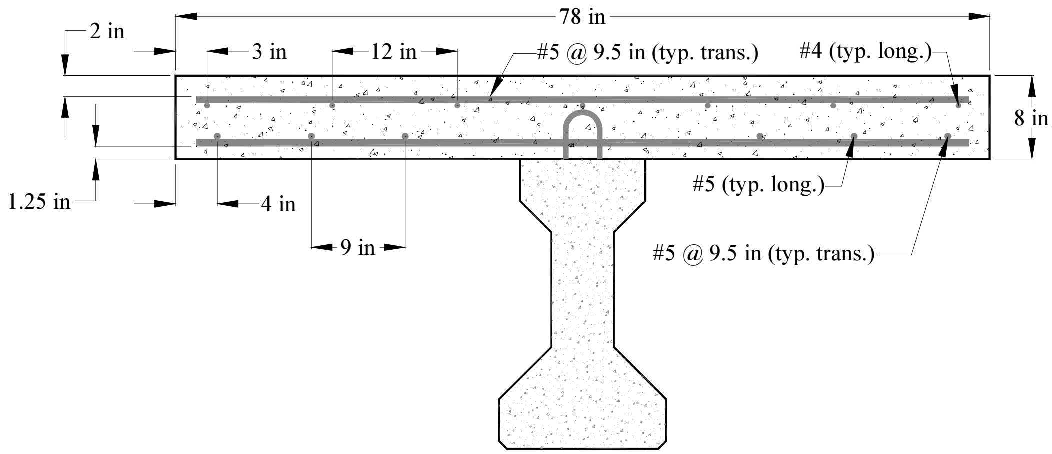

Figure 10.6 Concrete Deck Details without Panels

A normalweight concrete deck, a combination deck with lightweight panels and normalweight concrete, and a completely lightweight deck were built. The decks were each cast 8 in deep and 6 ft 6 in wide. The deck variations are shown in Table 10.3.

Table 10.3 Test Beam Concrete and Deck Type

Beam ID	Length	Beam Concrete	Deck Concrete	Lightweight Deck Panels
NW6000-1	40-ft	NW 6000-psi	NW 5000-psi	No
LW6000-1	40-ft	LW 6000-psi	NW 5000-psi	No
LW6000-2	40-ft	LW 6000-psi	NW 5000-psi	Yes
LW8000-1	40-ft	LW 8000-psi	NW 5000-psi	No
LW8000-2	40-ft	LW 8000-psi	NW 5000-psi	Yes
LW8000-3	40-ft	LW 8000-psi	LW 5000-psi	No

Notes:
LW = Lightweight
NW = Normalweight

The decks were all built in FSEL. The wood formwork was oiled before placement of the reinforcement to ease its removal upon curing of the concrete. The concrete was placed using an overhead bucket, was vibrated, and then finished with a screed, followed by bullfloats. Trowels were not used because the finish of the deck was not critical to the test procedure. The decks were allowed to cure for four days before formwork removal, and the beams did not undergo testing until the decks had reached their specified strength according to cylinder compression tests.

The precast panels used in two of the decks were cast at the Austin Prestressed Concrete Plant in Austin, Texas. They were shipped to FSEL and placed in the deck formwork using an overhead crane. The panels were placed on a ½ in thick by 1 in wide strip of fiberboard on the top edges of the beam. This allowed the greatest area of contact between the beam and the deck in order to develop full composite action, as shown in Figure 10.7. The reinforcement was slightly different in the decks with panels, as shown in Figure 10.8.

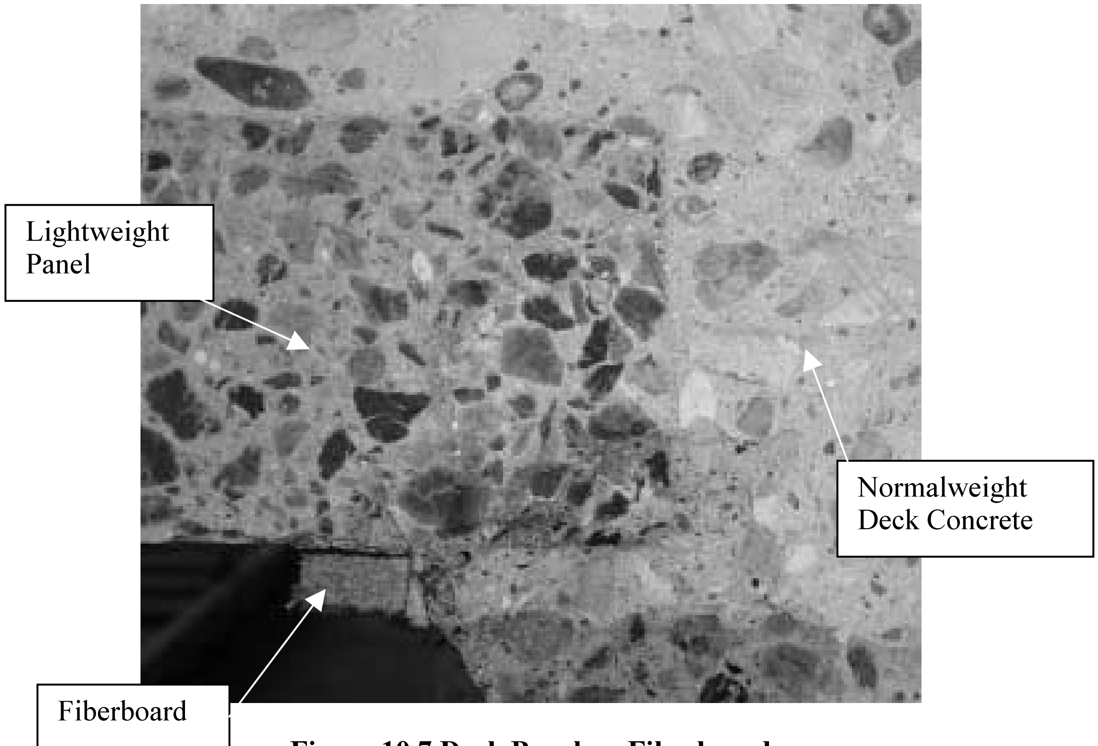

Figure 10.7 Deck Panel on Fiberboard

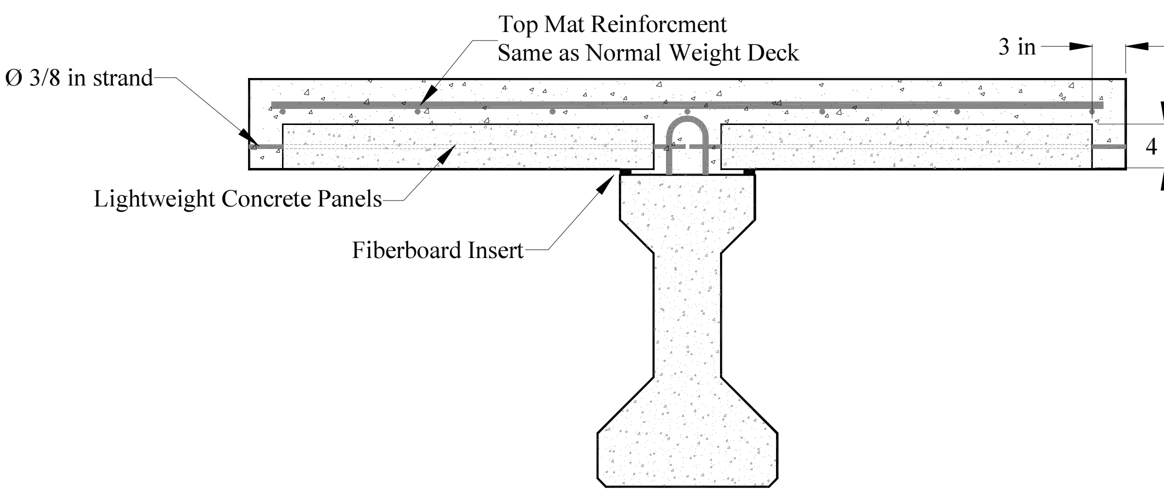

Figure 10.8 Concrete Deck Details with Panels

10.3 TEST SET-UP

10.3.1 General Layout

Each test beam was brought into FSEL using an overhead crane and placed on 11 in x 20 in x 3 in thick reinforced elastomeric bearing pads on top of 3 ft high reinforced concrete blocks. The centerline of the bearing pad was 6 in in from the end of the beam, as shown in Figure 10.9. The centerline of the opposing pad was 24 ft 6 in from the end of the beam. The beam was thus tested over a 24 ft span length, leaving a cantilevered length of the beam 15 ft 6 in long relatively unaffected by the test. Following a test, the geometry was reversed and the other end of the beam was tested. Using this approach, each beam could be tested twice, and the limited number of beams cast for the project was able to produce twice the results. This approach had been used with success on previous projects at FSEL for TxDOT.

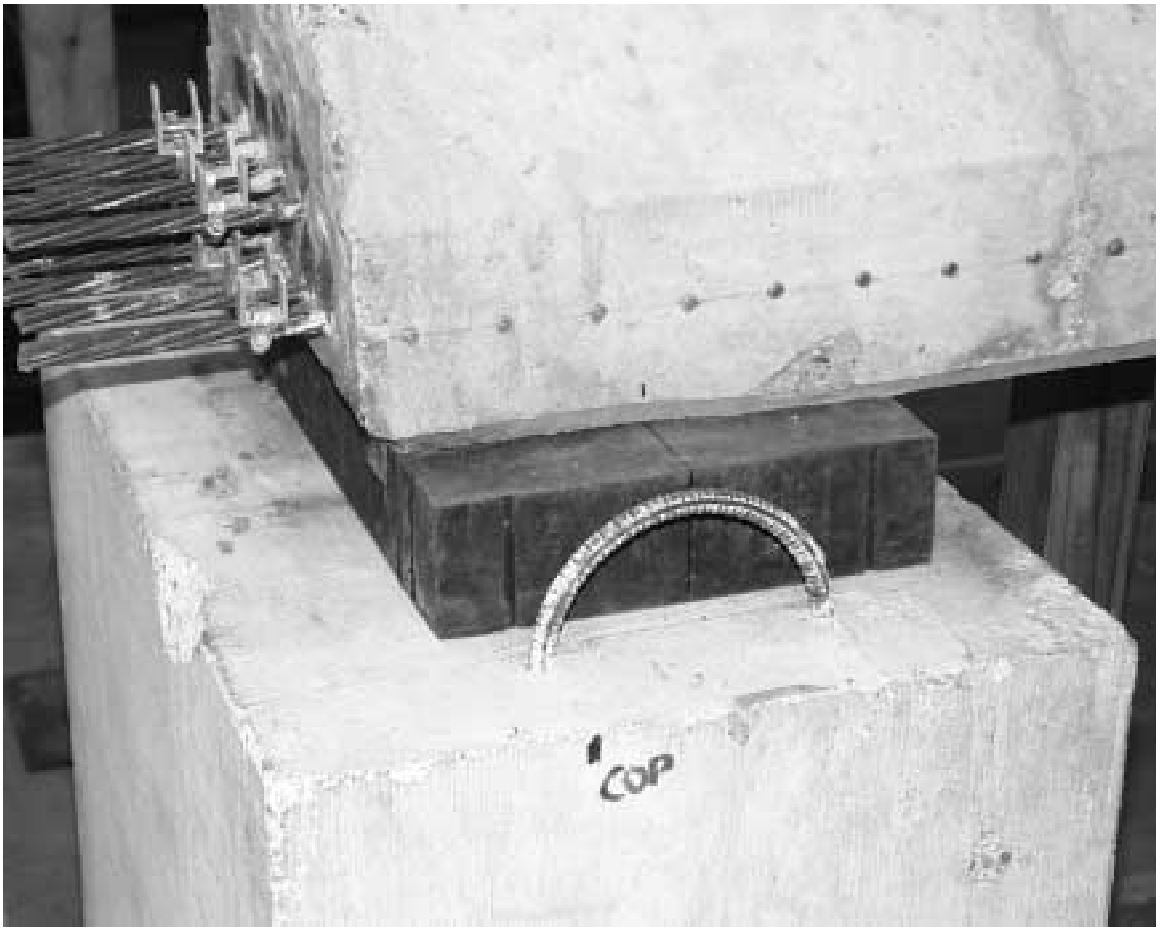

Figure 10.9 Beam on Pad and Concrete Block

10.3.2 Loading Apparatus

When the formwork from the deck was removed and the deck concrete was cured sufficiently to gain its desired strength, the load frame was brought into place. The load frame consisted of a four-column frame supporting a load actuator on a steel I-beam, as shown in Figure 10.10.

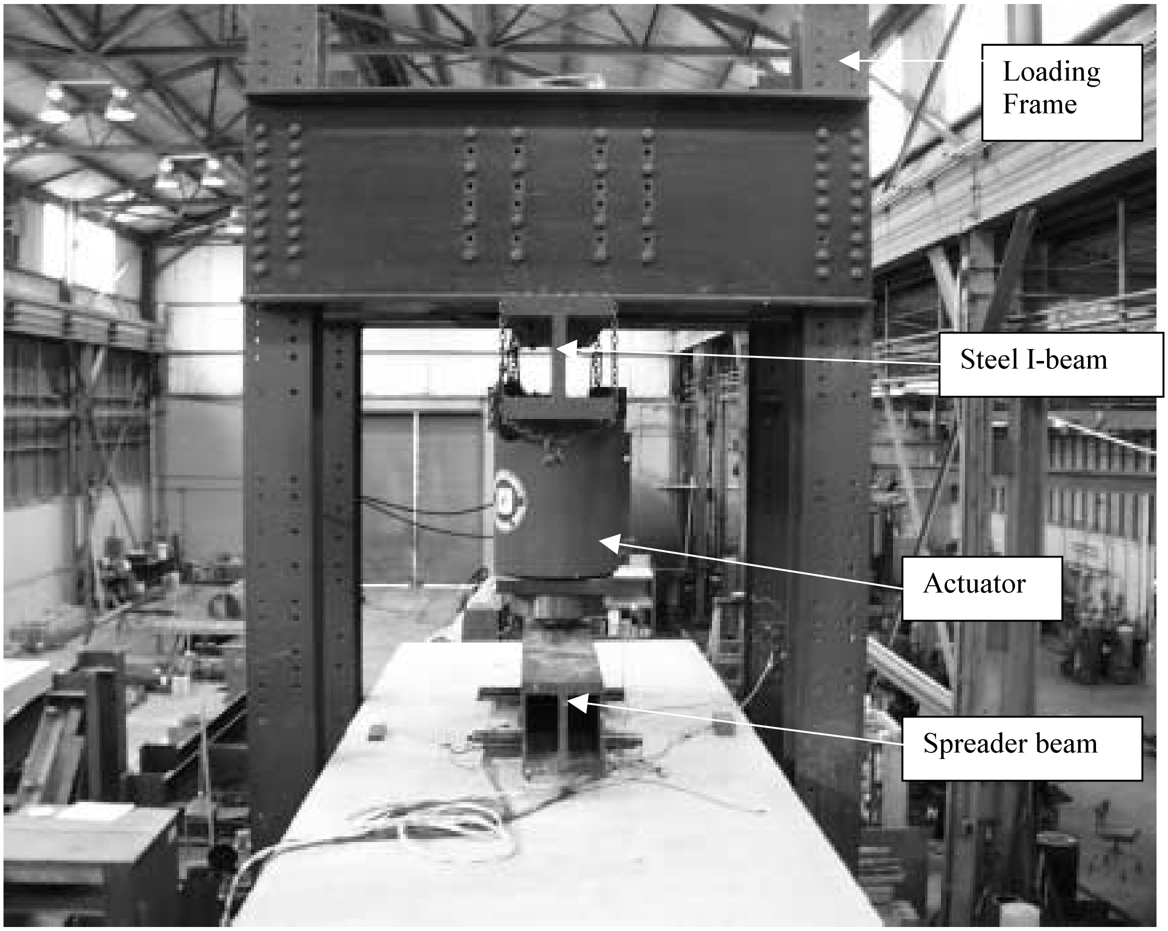

Figure 10.10 Load Frame and Actuator

The I-beam allowed the actuator to be moved longitudinally along the beam. The load was transferred from the load actuator into a spreader beam placed on top of two 2.5 in diameter steel roller supports. The rollers sat on two steel loading pads and were free to roll, thereby preventing the transfer of any horizontal shear into the test beam. The steel pads were hydrostoned to the top of the deck to prevent excess movement. The frame itself was firmly bolted down to a structural floor in order to prevent additional excess movement during testing. The spreader beam, rollers, and steel pads are shown in Figure 10.11.

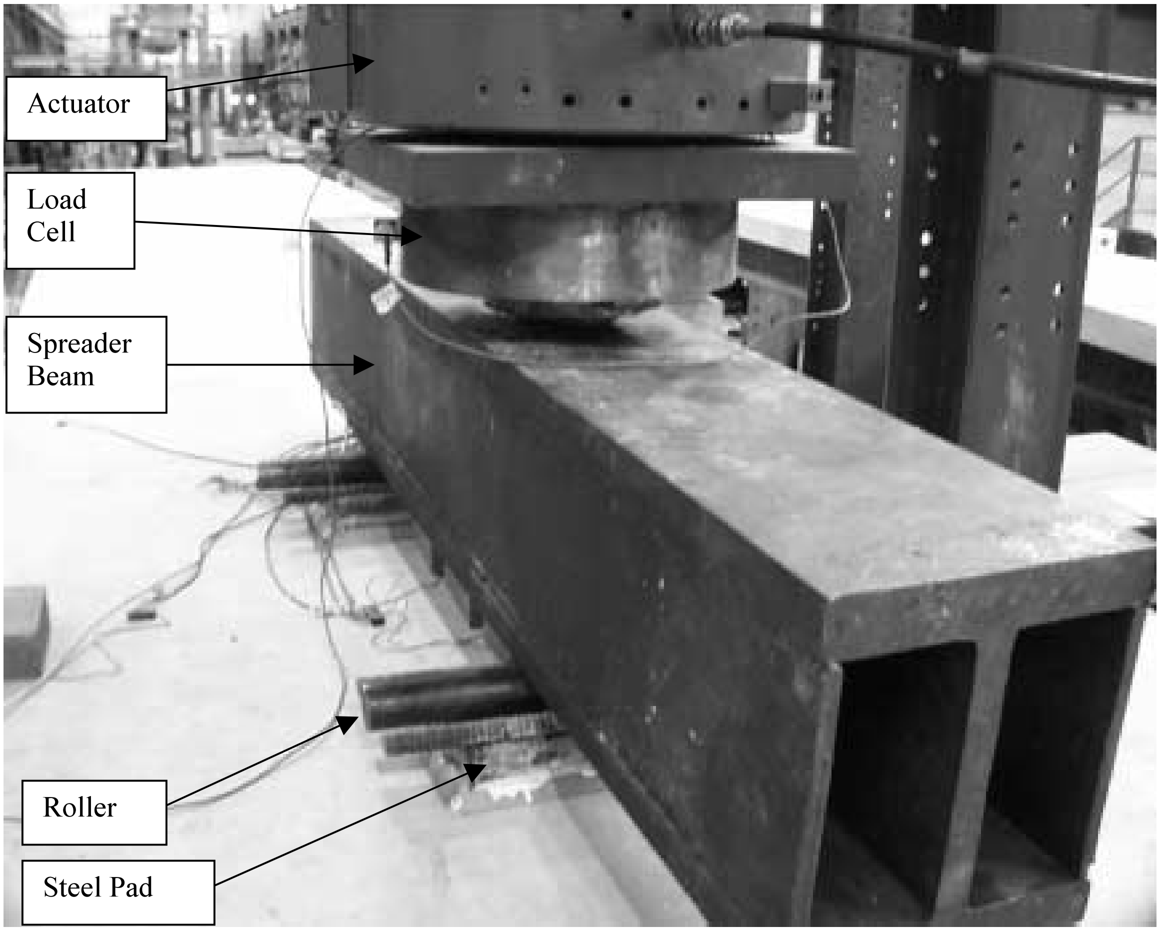

Figure 10.11 Spreader Beam, Rollers, and Loading Pads

The load was applied to the test beam at two contact points. Due to the geometry of the setup, it was possible to position the actuator at a point along the spreader beam such that the two loads applied to the beam through the loading pads would create a constant moment region in the bottom flange of the test beam directly underneath the loading pads. The exact point for the actuator on the spreader beam varied for the different embedment lengths being tested, as well as the concrete deck composition. The loading pads were placed such that the first loading pad was a distance from the end of the beam equal to the embedment length, L_e, being tested. This configuration is shown in Figure 10.12. The variation of the embedment lengths and the actuator load points on the spreader beam are listed in Table 10.4.

The embedment lengths for each test were chosen in such a way as to develop the greatest amount of meaningful information with the limited number of tests available. The predicted development length per AASHTO [1] was calculated as 82 in for the normalweight beam. It was decided that the first test should occur at an embedment length of 80 in, since previous experience had indicated that the AASHTO development length was conservative. The shortest embedment length that could be tested and still produce a flexural failure while not risking a sudden shear failure was 60 in. The normalweight beam was then tested at both 80 and 60 inches. The subsequent lightweight beams were then tested at 60, 70, and 80 in, in order to observe whether a bond failure would occur and if any strands would slip.

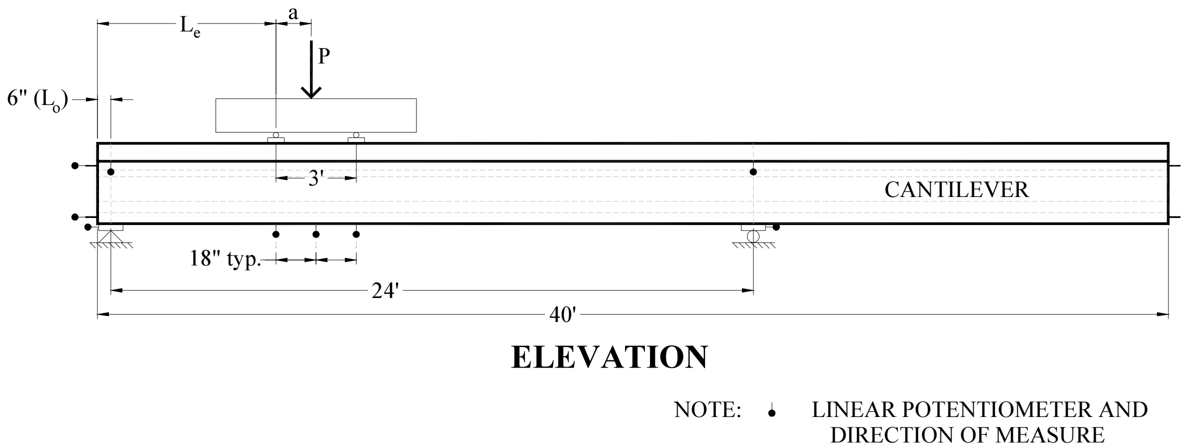

Figure 10.12 Test Set-up Geometry

Table 10.4 Test Configurations

Beam ID	Embedment Length,* in	a* in	Lightweight Deck Panels
NW6000-1-N-80	80	10.8	No
NW6000-1-S-60	60	9.18	No
LW6000-1-N-80	80	10.60	No
LW6000-1-S-70	70	9.12	No
LW6000-2-N-70	70	9.12	Yes
LW6000-2-S-60	60	7.68	Yes
LW8000-1-N-80	80	10.60	No
LW8000-1-S-70	70	9.12	No
LW8000-2-N-70	70	9.12	Yes
LW8000-2-S-60	60	7.68	Yes
LW8000-3-N-70	70	9.20	LW deck
LW8000-3-S-60	60	7.62	LW deck

***see Figure 10.12 for dimensions "a" and "L_e"**

10.3.3 Instrumentation

The instrumentation used in this test procedure was electronic, with selected manual measurements made to check the accuracy of the electronic results. All of the data-collecting instruments were attached by wires to a bridge box, from which cables carried the information into the data acquisition device. This device was connected to a computer running a program that managed the data. The types of measurement devices and instrumentation are described in the following sections.

10.3.3.1 Load Measurement

The measurement of the amount of load applied to the beam was carried out in both manual and electronic form. A pressure gauge was attached to the hydraulic pump operating the actuator, as shown in Figure 10.13. A reading of this gauge was taken manually at each load increment during the test procedure. This was compared to the measurement taken by the load cell attached to the actuator and wired into the data acquisition system. The load cell data was taken and recorded continuously throughout the test procedure.

Figure 10.13 Hydraulic Pump and Pressure Gauge

10.3.3.2 Beam Displacement Measurement

The vertical displacement of the beam was taken at three points below the beam. The first point was located at a distance from the end of the beam equal to the embedment length being tested. The second point was 18 in (457.2 mm) beyond the first, and the third was at a point 36 in (0.9 m) beyond the first, directly beneath the second loading pad. The displacement was measured using 6 in (152.4 mm) linear potentiometers placed in a frame and wired to the data acquisition system, as shown in Figure 10.14. A glass slide was attached to the bottom of the beam at the point where the potentiometer came into contact with the beam to ensure a smooth, consistent testing surface.

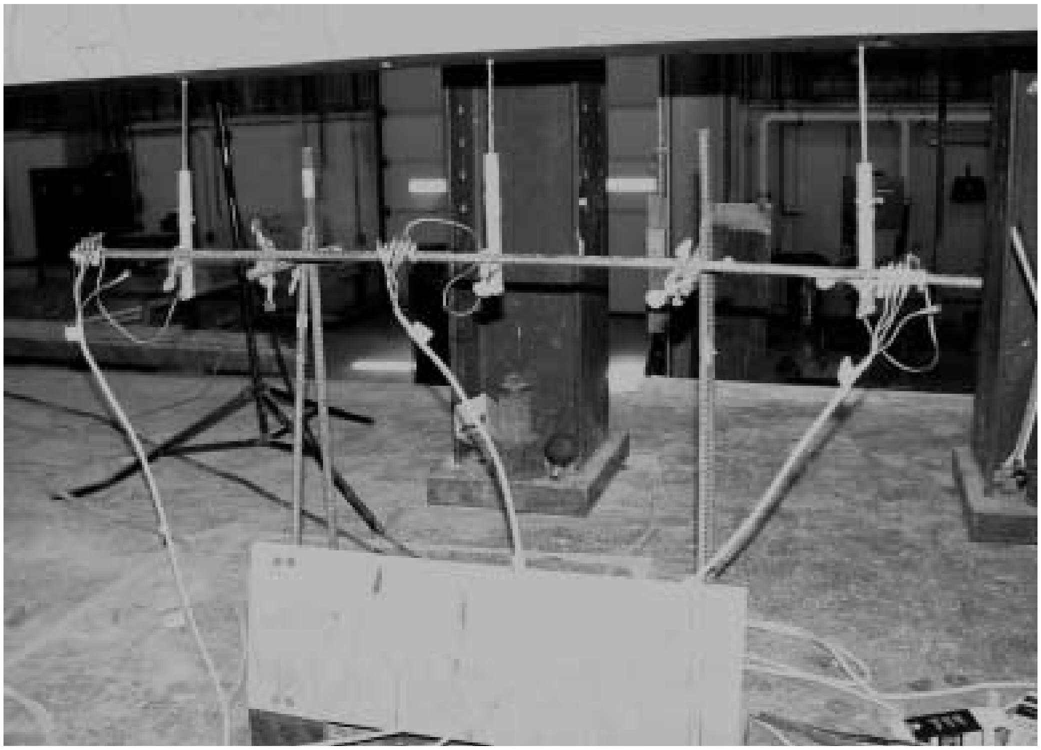

Figure 10.14 Vertical Displacement Potentiometers

As a manual backup to this system, a ruler was attached to side of the beam directly above the location of the second potentiometer. A piano wire was stretched along the beam from support to support, and attached at a level equivalent to the centroid of the cross section. Thus, as the beam deflected and rotated at the supports, the vertical position of the piano wire attachment points remained constant. A reference measurement was taken on the ruler before the test began, and then manual measurements of where the wire crossed the ruler were taken at each load increment during the test procedure, similar to the readings of the pressure gauge previously mentioned.

10.3.3.3 Support Displacement Measurement

The displacement of the support was taken using several types of linear potentiometers at different points. The horizontal displacement of the bearing pad was measured using 2 in potentiometers placed in a block support and wired to the data acquisition system. These are shown in Figure 10.15. A glass slide was attached to the beam at the point where the potentiometer came into contact with the beam to ensure a smooth, consistent testing surface. The vertical displacement of the support was measured using 5 in string potentiometers attached to the bottom of the deck and to the concrete support block, as shown in Figure 10.16. The same arrangement was used at each support location of the beam during a test.

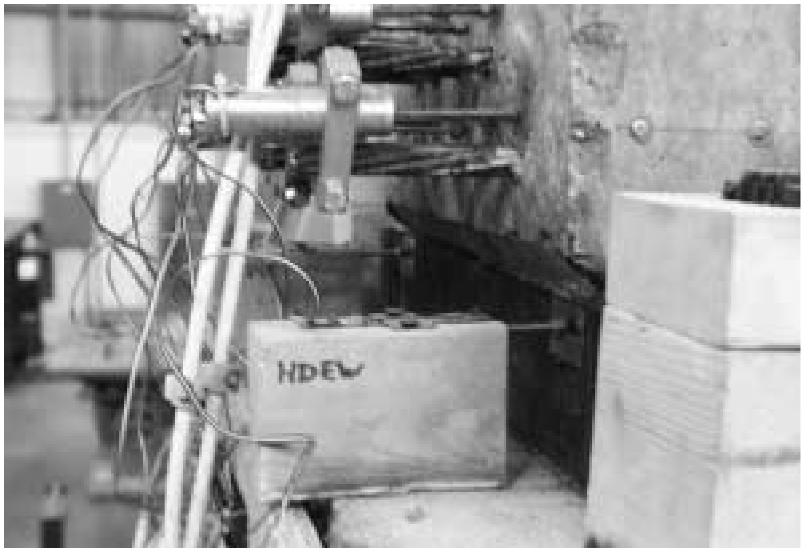

Figure 10.15 Horizontal Support Displacement Potentiometer

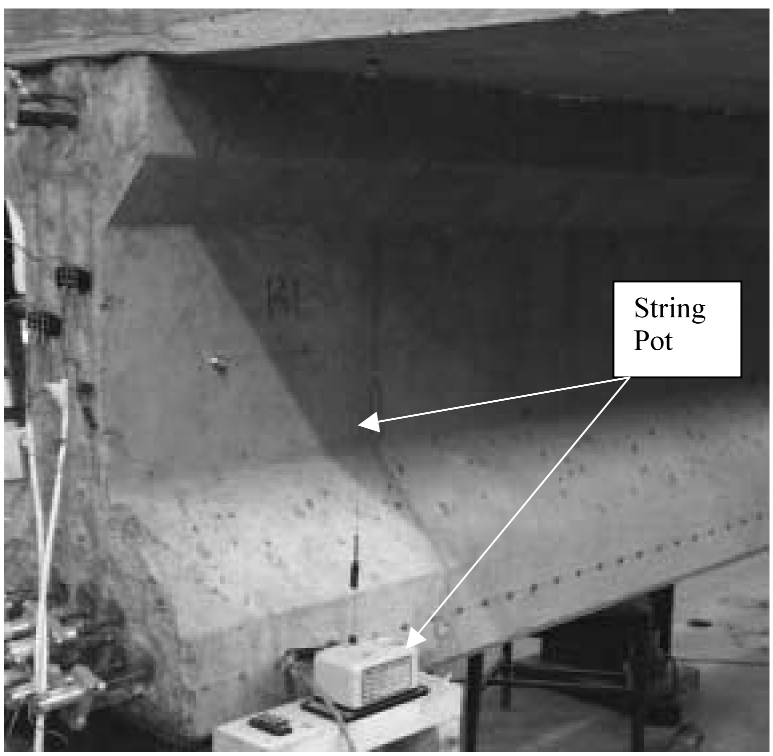

Figure 10.16 Vertical Support Displacement Potentiometer

10.3.3.4 Strand Slip Measurement

The measurement of the slip, or bond failure, of the prestressing strands was a critical part of the test procedure, since the slip of a strand would be an indication of a limit on the development length of the beam. The measurements were taken on each strand using a 2 in linear potentiometer that was attached to the strands using a bracket system as shown in Figure 10.17.

A glass slide was attached to the beam at the point where the potentiometer came into contact with the beam to ensure a smooth, consistent testing surface. The potentiometers and brackets were adjusted such that each was free to move any distance in or out relative to the others, allowing complete measurement of the potential slip of the strand. They were then wired into the data acquisition system. The wiring is shown in Figure 10.18.

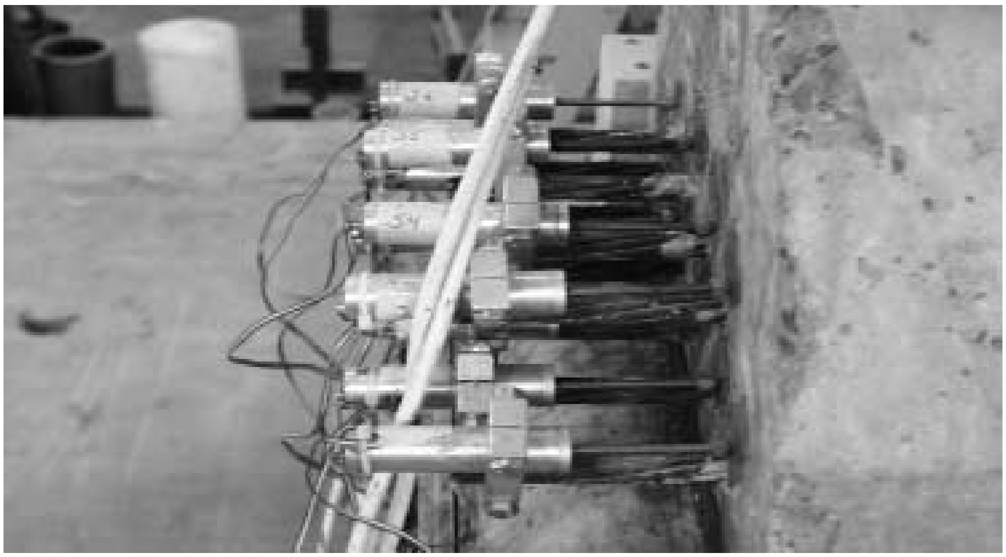

Figure 10.17 Strand Slip Measurement

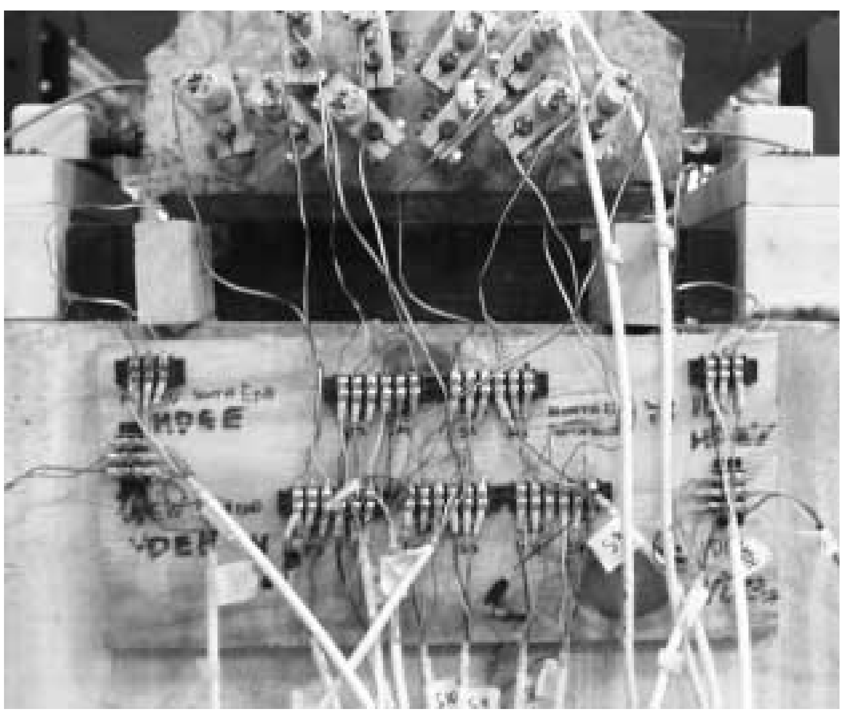

Figure 10.18 Strand Potentiometer Wiring

10.3.3.5 Strain Measurement

The top fiber strain of the concrete deck was measured using eight strain gauges placed in the arrangement shown in Figure 10.19. Prior to placing the strain gauges, the surface of the deck concrete was carefully prepared. The gauging area was first ground down to exposed aggregate, and then treated with several chemicals before a layer of epoxy was placed. After allowing the epoxy to set for a day, the epoxy was treated with chemicals and then the gauge was attached, using the same epoxy. It was held down using a typical brick. Between the gauge and the brick, however, a soft pad was placed to prevent damage to the gauge. These pads are visible in Figure 10.20. The gauges were then wired and connected to the data acquisition system.

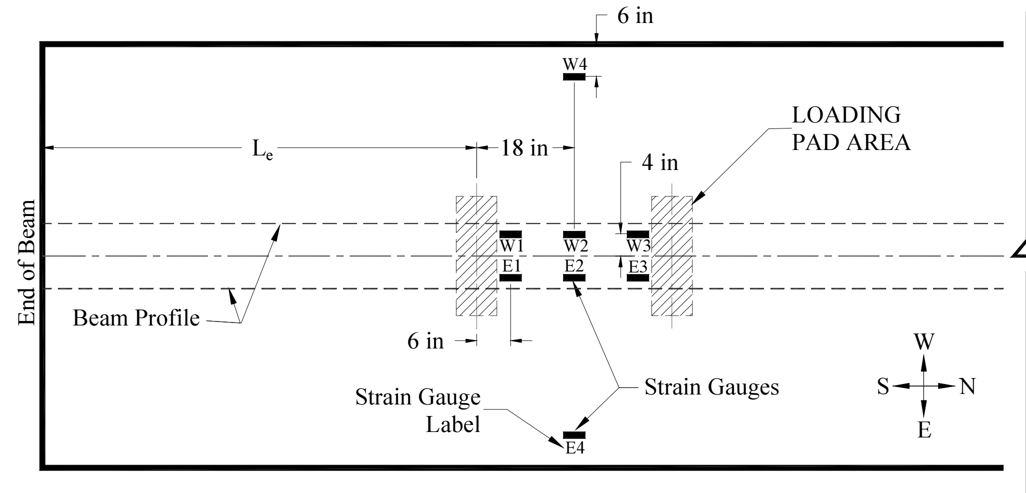

Figure 10.19 Strain Gauge Locations

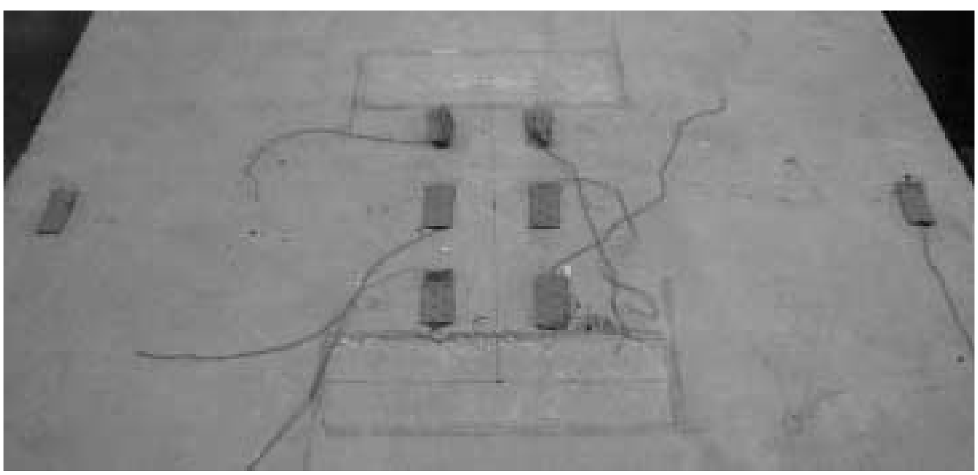

Figure 10.20 Strain Gauge Locations and Pads

10.3.3.6 Crack Measurements

The sizes of the cracks were measured at the ultimate stage of loading during the test procedure. This was done manually, using a crack width indicator.

10.3.4 Data Acquisition Equipment

The data acquisition system consisted of bridge boxes, wires, cables, and a computer system to handle the data, as shown in Figures 10.21 and 10.22. The computer ran an Excel program called Measure that managed the data acquisition. The system took data readings from all potentiometers and strain gauges every 5 seconds during loading. The data was taken every 10 seconds while load application was paused and cracks were marked. Following the test, the data was reduced for some calculations to the key points corresponding with the load increments and any significant events that occurred. The full data set was used for certain calculations as well.

Figure 10.21 Data Acquisition System

Figure 10.22 Data Acquisition Computer

10.4 TEST PROCEDURE

10.4.1 Loading

The test procedure consisted of several stages. The loading took place in approximately 30 kip increments until first cracking occurred. The cracking load was noted, and then the loading continued in 30 kip increments. At each load increment, manual readings of the pressure gauge and the vertical displacement of the beam were made, and cracks were marked. Continuously updated graphs of the strand slip, the load versus concrete deck strain, and the load versus vertical displacement of the beam were monitored throughout the entire test procedure. After the failure limit was reached, the beam was unloaded and final measurements were taken to observe the inelastic effects on the beam.

10.4.2 Cracking

At each load increment, time was taken to mark all of the cracks visible on the beam at that stage. Flexural cracks and shear cracks were marked in separate colors. At high loads, where flexural-shear cracks develop, the cracks were all marked in the flexural crack color. After marking all of the cracks, pictures were taken of the current crack pattern. Pictures were also taken following any significant events. At the failure limit, the widths of the largest cracks were measured and noted.

10.4.3 Failure Limit

The failure limit was slightly different for each test. Careful attention was given to the strain in the top fiber of the deck concrete and the vertical displacement of the beam. Crushing failure of the deck concrete was the desired failure limit, but several of the tests were halted short of that mark due to safety concerns over high strains and/or limiting deflections.

CHAPTER 11: DEVELOPMENT LENGTH TEST RESULTS

11.1 INTRODUCTION

This chapter covers the testing of the nominal 6000 psi beam series carried out by Kolozs [43], and the testing of the nominal 8000 psi beam series carried out by Thatcher [75]. Test results covering multiple categories of behavior are presented and compared, and the results are discussed.

11.2 TEST RESULTS

This section covers the results from the testing of both the 6000 psi and 8000 psi beam series. The material properties of the individual beams, decks, and panels are presented, followed by the results from the tests. The categories of behavior reported include initial stiffness, cracking and ultimate moment and load, strand elongation, maximum strain and displacement, crack patterns, strand slip, and failure types. A plot of the applied moment versus displacement of all six tests carried out by Thatcher is shown in Figure 11.1, and a plot of the applied moment versus displacement of all six tests carried out by Kolozs is shown in Figure 11.2. These figures provide an overall perspective of the test results before the details are given. The curves show that the behavior of the six beams during the twelve tests was very similar, and the stiffnesses and moments achieved show good correlation of the data.

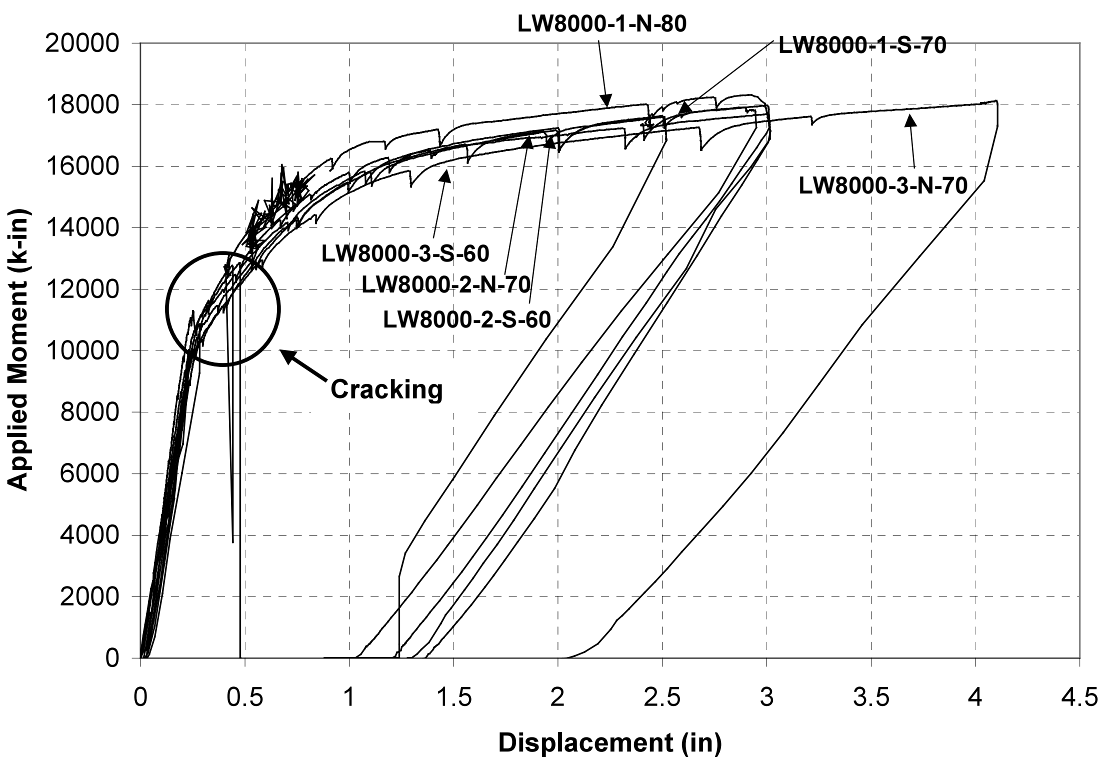

Figure 11.1 Initial Results - Thatcher

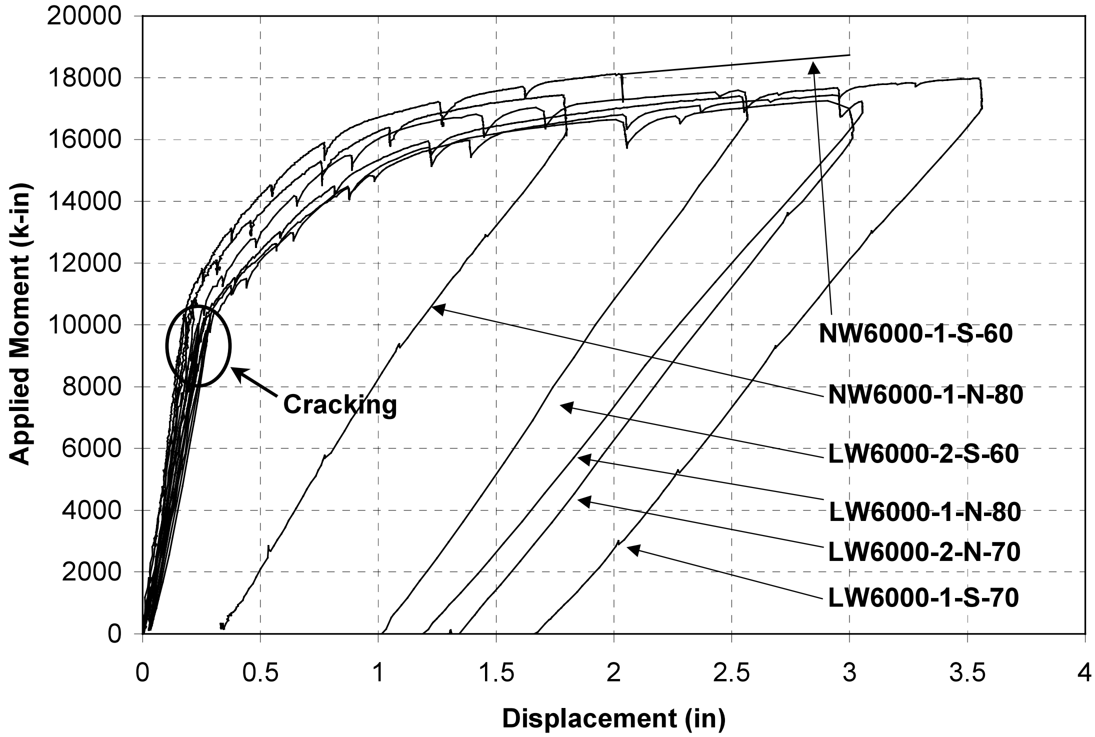

Figure 11.2 Initial Results - Kolozs

11.2.1 Beam Properties

The beams tested consisted of nominal 6000 psi compressive strength normalweight concrete, nominal 6000 psi compressive strength lightweight concrete, and nominal 8000 psi compressive strength lightweight concrete. The beam properties are given in Table 11.1. The normalweight beam had a normalweight cast-in-place concrete deck. Of the 6000 psi lightweight concrete beams, the first had a normalweight cast-in-place concrete deck, and the second a combined deck of lightweight concrete panels and normalweight cast-in-place concrete. Of the 8000 psi beams, the first had a normalweight cast-in-place concrete deck, the second a combined deck of lightweight concrete panels and normalweight cast-in-place concrete, and the third a cast-in-place lightweight concrete deck.

Table 11.1 Beam Properties

Beam	L_e (in)	Beam Concrete Strength (psi)	Beam Modulus (ksi)	Deck Concrete Type	Deck Strength (psi)	Deck Modulus (ksi)	Panels	Panel Strength (psi)	Panel Modulus (ksi)
NW6000-1-N	80	6607	5742	NW	5399	5335	No		
NW6000-1-S	60	6772	5335	NW	5679	5076	No		
LW6000-1-N	80	8126	3253	NW	5285	5335	No		
LW6000-1-S	70	8030	3253	NW	5285	5335	No		
LW6000-2-N	70	8030	3253	NW-LWP	4551	4486	Yes	7235	2539
LW6000-2-S	60	8030	3253	NW-LWP	4551	4486	Yes	7235	2539
LW8000-1-N	80	6848	2576	NW	5112	4803	No		
LW8000-1-S	70	6848	2576	NW	5112	4803	No		
LW8000-2-N	70	7847	3141	NW-LWP	5182	4453	Yes	7321	2680
LW8000-2-S	60	7847	3141	NW-LWP	5182	4453	Yes	7321	2680
LW8000-3-N	70	8013	3104	LW	7011	2511	No		
LW8000-3-S	60	8013	3104	LW	7011	2511	No		

The actual concrete strength of the normalweight pretensioned beam at the time of testing was well above (10%) the specified 6000 psi. The actual concrete strength of the two lightweight 6000 psi beams at the time of testing was significantly higher (33%) than the specified strength. The actual concrete strength of the first of the three 8000 psi beams at the time of testing was well below (15%) the specified 8000 psi strength, the actual strength of the second was slightly below (2%) the specified 8000 psi strength, while the third beam had an actual concrete strength just at 8000 psi.

The modulus of elasticity of the 6000 psi normalweight beam concrete was significantly higher (70%) than the moduli of the 6000 psi lightweight beams in which the actual concrete strength was over 8000 psi. This was to be expected. The moduli of elasticity of the lightweight concrete in the three 8000 psi concrete beams were relatively similar to the LW 6000 beams when actual f'_c was considered.

The strengths of the normalweight concrete decks were very similar, as well as the normalweight concrete used in LW8000-2 with the panels, and were close (6%) to the specified 5000 psi strength. The strength of the normalweight concrete deck on LW6000-2 was a bit lower (9%) than desired. The lightweight concrete used for the deck on LW8000-3 was specified to be 5000 psi strength, but when tested was around 40% stronger at 7000 psi.

The moduli of elasticity of the normalweight cast-in-place concrete decks were similar in both the 6000 psi beams and the 8000 psi beams, while the modulus of the deck concrete on LW6000-2 was 10% lower, and the modulus of the lightweight concrete cast-in-place deck on LW8000-3 was significantly less than the other decks, but similar to that of the lightweight beams. The lightweight concrete panels had achieved a strength slightly (4%) above 7000 psi when the beam was tested, with a modulus similar to that of the full-depth cast-in-place lightweight concrete deck.

The dead load weight (lb/ft) of the composite concrete beams varied according to the amount of lightweight concrete in the composite beam. Figure 11.3 shows the dead load weight of three normalweight precast concrete beams with varying deck components, and three lightweight precast concrete beams with varying deck components. The composition of each beam is indicated by the figure above each bar, with white used to indicate normalweight concrete and black used to indicate lightweight concrete. The actual test program included all three lightweight concrete beam combinations shown, but only the leftmost normalweight concrete beam and slab. The other two normalweight concrete beam combinations are presented only for comparison. The reduction in weight due to the lightweight concrete can be particularly significant when dealing with long spans, where the savings in dead load may allow the use of fewer strands, smaller sections, fewer beams in a particular bridge design, or reduced crane capacities.

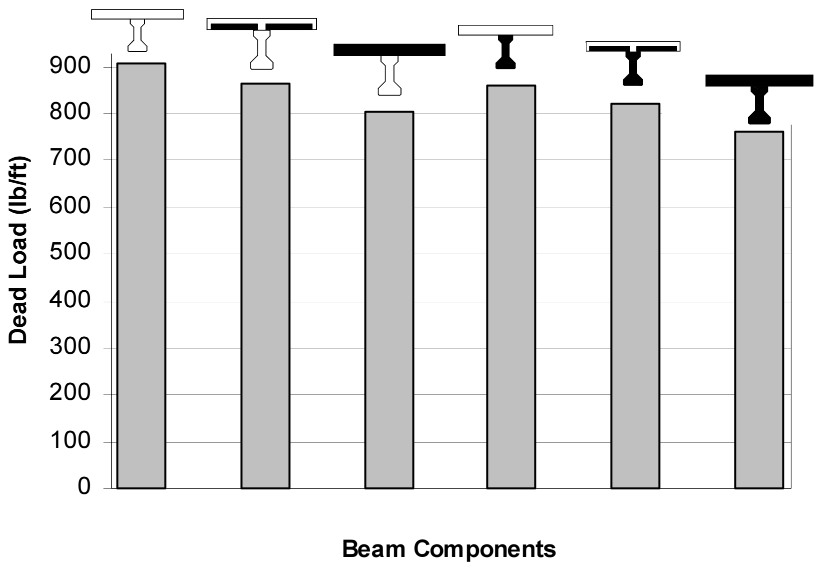

Figure 11.3 Dead Load

The twelve tests took place at three different embedment lengths in order to experimentally determine the development length of the beams. These ranged from 80 to 60 in (2032 to1524 mm), with a similar length repeated between beams in order to allow correlation of the test data from the different beams. The normalweight concrete beam was tested at the two bounds of embedment length testing, 80 and 60 in (2032 to 1524 mm), in order to bracket the testing of the lightweight concrete beams.

11.2.2 Initial Stiffness

The initial stiffnesses (EI) of the beams are presented in this subsection. The stiffnesses calculated as the product of the gross section moment of inertia calculated from the geometry of the cross section and the measured concrete modulus of elasticity are shown in Table 11.2 and Figure 11.4. The initial stiffnesses determined from the load-deflection relationships during the testing of the beams are also provided. Table 11.1 indicates that the lightweight concrete used in the beams had a modulus of elasticity around 55% that of the normalweight concrete. The composite beams tested had varying components of normalweight and lightweight concrete. One composite beam was a normalweight concrete beam with a normalweight concrete deck. Of the five lightweight concrete composite beams, while all had precast beams of lightweight concrete, the amount of lightweight concrete in the deck varied. This is indicated by the figures above each pair of bars in Figure 11.4, with lightweight concrete indicated by the black portions of the cross section.

Table 11.2 Initial Stiffness

Beam ID	Modulus of Elasticity (ksi)			Moment of Inertia (in^4) Transformed to E_{beam}	Calculated Stiffness ($k\text{-}in^2$)	Measured Stiffness ($k\text{-}in^2$)	Ratio Measured / Calculated
	E_{beam}	E_{Deck}	E_{Panel}				
NW6000-1-N-80	5742	5335		102,202	5.87E+08	1.39E+09	2.36
NW6000-1-S-60	5335	5076		102,769	5.48E+08	1.23E+09	2.24
LW6000-1-N-80	3253	5335		120,272	3.91E+08	1.03E+09	2.63
LW6000-1-S-70	3253	5335		120,272	3.91E+08	1.07E+09	2.74
LW6000-2-N-70	3253	4486	2539	111,119	3.61E+08	1.60E+09	4.43
LW6000-2-S-60	3253	4486	2539	111,119	3.61E+08	1.14E+09	3.15
LW8000-1-N-80	2576	4803		125,979	3.25E+08	1.26E+09	3.88
LW8000-1-S-70	2576	4803		125,979	3.25E+08	1.22E+09	3.75
LW8000-2-N-70	3141	4453	2680	112,019	3.52E+08	--	--
LW8000-2-S-60	3141	4453	2680	112,019	3.52E+08	8.32E+08	2.36
LW8000-3-N-70	3104	2511		99,147	3.08E+08	1.03E+09	3.34
LW8000-3-S-60	3104	2511		99,147	3.08E+08	8.03E+08	2.60

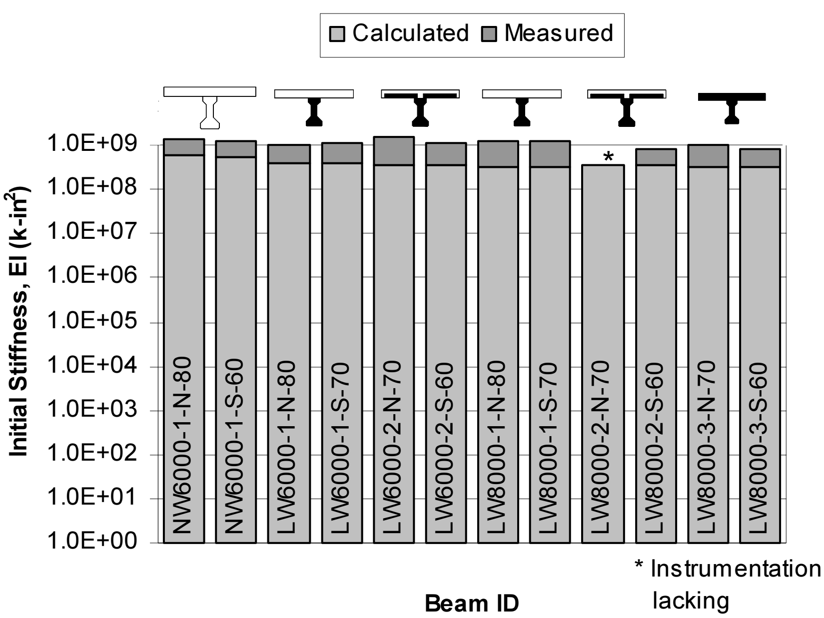

Figure 11.4 Initial Stiffness

The results of the testing indicate that the expectation of slightly lower initial stiffness for increased percentage of lightweight components in the cross section was generally true. However, the relative effect was small when considering the magnitude of the values. The measured initial stiffnesses were 220 to 440% higher than the calculated values, but showed the predicted trend of decreasing stiffness with increased lightweight concrete components. All of the beams had relatively similar initial stiffnesses, and show good correlation of the test data. The reduced stiffness of lightweight concrete should not be a significant problem in pretensioned girder applications.

11.2.3 Cracking and Ultimate Moment and Load

The cracking and ultimate moments are shown in Figure 11.5. The ultimate moments are relatively the same for all of the tests, as was expected due to the similarity of the beam cross sections and the known fact that there is no essential difference in application of the rectangular stress block with normalweight and lightweight concrete. The cracking moments show greater differences. This is partially due to the difficulty in observing the exact onset of flexural cracking in the beam during testing and partially because of the lower split cylinder tensile strength of the lightweight concrete.

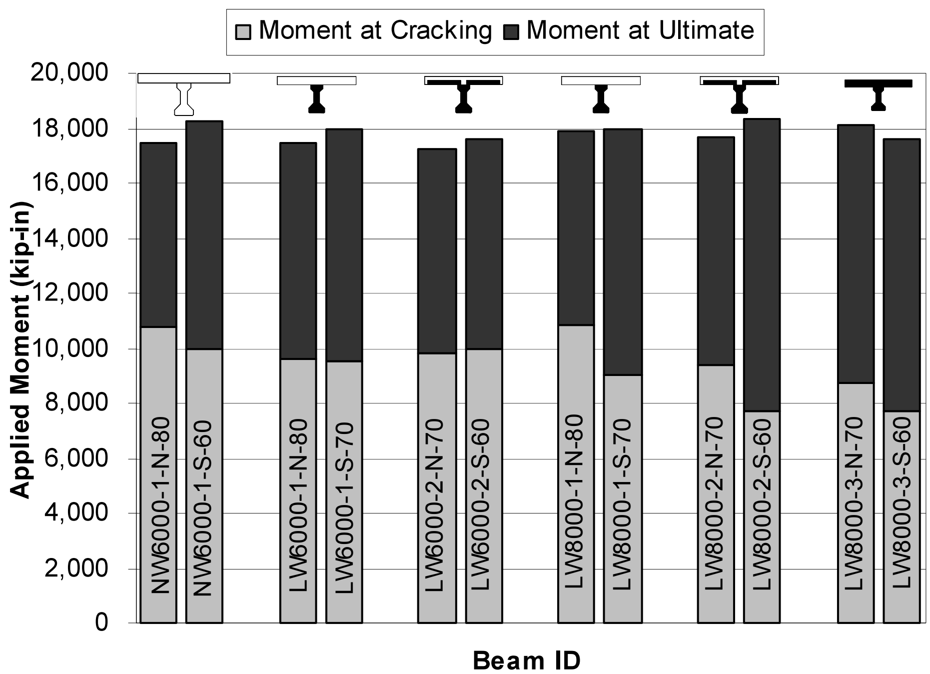

Figure 11.5 Cracking and Ultimate Moments

11.2.4 Strand Elongation

The elongation of the strands at ultimate is presented in this subsection. The strand elongation was calculated for the bottom layer of strands in the beams. The values were obtained assuming a linear strain distribution in the composite beams. The depth of the cracking in the deck was observed visually, and the maximum strains in the deck above the web due to the load were measured. The strain in the steel was then calculated from the deck strain and the geometry of the cross section. This was added to the strain already present in the strands due to prestressing and subsequent losses. From this final strain in the steel strands, the elongation was calculated. The results are given in Table 11.3 and shown in Figure 11.6. The data of LW8000-2-N-70 is incomplete, due to problems with the strain gages during that particular test.

Table 11.3 Calculated Strand Elongation at Ultimate

Beam ID	L_e in	Depth of Cracking at Ultimate, in	Concrete Strain at Top, microstrain	Strand Elongation (%)
NW6000-1-N	80	2.7	2,033	2.9
NW6000-1-S	60	3.0	2,688	3.3
LW6000-1-N	80	2.5	4,349	6.0
LW6000-1-S	70	2.8	3,266	4.2
LW6000-2-N	70	1.9	3,462	6.4
LW6000-2-S	60	1.4	4,015	9.6
LW8000-1-N	80	2.9	3,650	4.5
LW8000-1-S	70	2.5	2,732	4.0
LW8000-2-N	70	2.5	--	--
LW8000-2-S	60	2.5	3,061	4.4
LW8000-3-N	70	2	3,711	6.5
LW8000-3-S	60	2.5	3,646	5.1

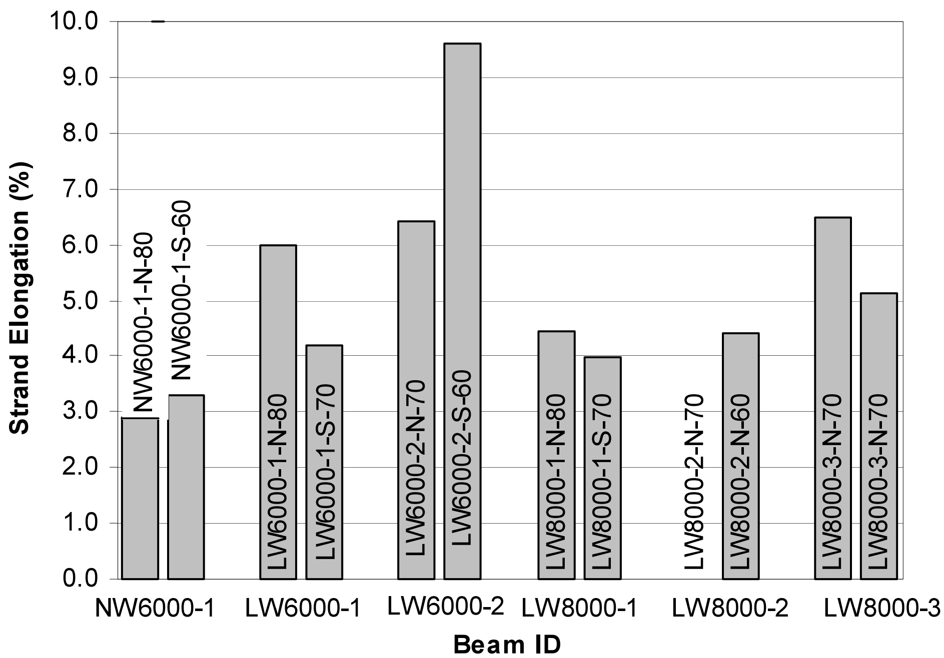

Figure 11.6 Calculated Strand Elongation at Ultimate

Yielding of the strands will occur at around 1% elongation, and the percentage strain in the strands at ultimate for each of the tests was substantially higher than that value. Between 1% and 4% elongation, the individual strand will undergo an increase in load of about 2.3 kips. The large yielding of the strands at ultimate is an indication that the flexural ultimate load was actually reached, since after yielding of the strands, further increases in strain occur without significant increases in stress, or load level. Only one of the strands slipped during the testing of the beams, LW6000-2-S-60, and the likely cause of the slip was suggested by Kolozs to be form oil accidentally placed on the strand when the beam was cast. [43]

11.2.5 Maximum Strain and Displacement

The average measured deck strains and maximum deflections are given in Table 11.4. Due to shear lag across the deck, the average of the strain gage readings was used. The data for LW8000-2-N-70 is incomplete, due to problems with the strain gages during that particular test. The maximum deflections were taken at the midpoint of the constant moment region in the beam.

Table 11.4 Strains and Deflections

Beam	Le	Maximum Load (kips)	Average Strain (microstrain)	Maximum Deflection (in)
NW6000-1 N	80 in	328	1697	1.8
NW6000-1 S	60 in	425	2105	3.0
LW6000-1 N	80 in	329	2755	3.0
LW6000-1 S	70 in	375	2824	3.5
LW6000-2 N	70 in	360	2958	2.9
LW6000-2 S	60 in	409	2651	2.5
LW8000-1 N	80 in	335	2695	2.9
LW8000-1 S	70 in	375	1984	3.0
LW8000-2 N	70 in	365	---	3.0
LW8000-2 S	60 in	425	2208	2.9
LW8000-3 N	70 in	375	3400	4.0
LW8000-3 S	60 in	410	2590	2.5

Plots of the applied moment vs. average microstrain across the top of the deck for each test are shown in Figures 11.7 and 11.8, for both Thatcher's [75] and Kolozs' test data [43]. The curves show similar behavior from all six tests within a beam series, as well as those carried out by Thatcher versus those carried out by Kolozs. Plots of the applied load vs. average microstrain across the top of the deck for each test are shown in Figures 11.9 and 11.10, for both Thatcher's [75] and Kolozs' test data [43]. The curves for tests of the same embedment length show similar behavior, as do the tests carried out by Thatcher versus those carried out by Kolozs. Individual plots of the load vs. microstrain readings for each test are given in References 43 and 75.

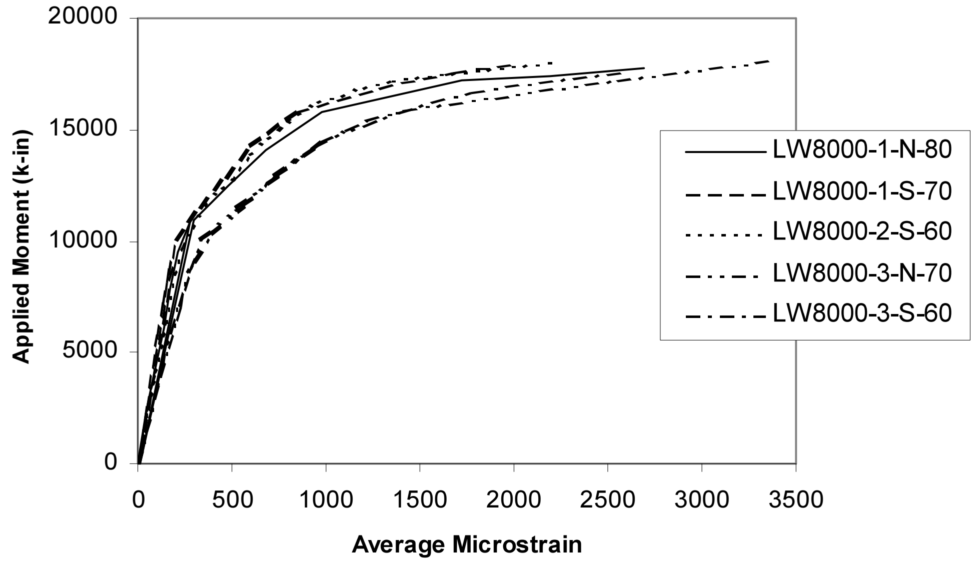

Figure 11.7 Applied Moment vs. Average Microstrain – Thatcher

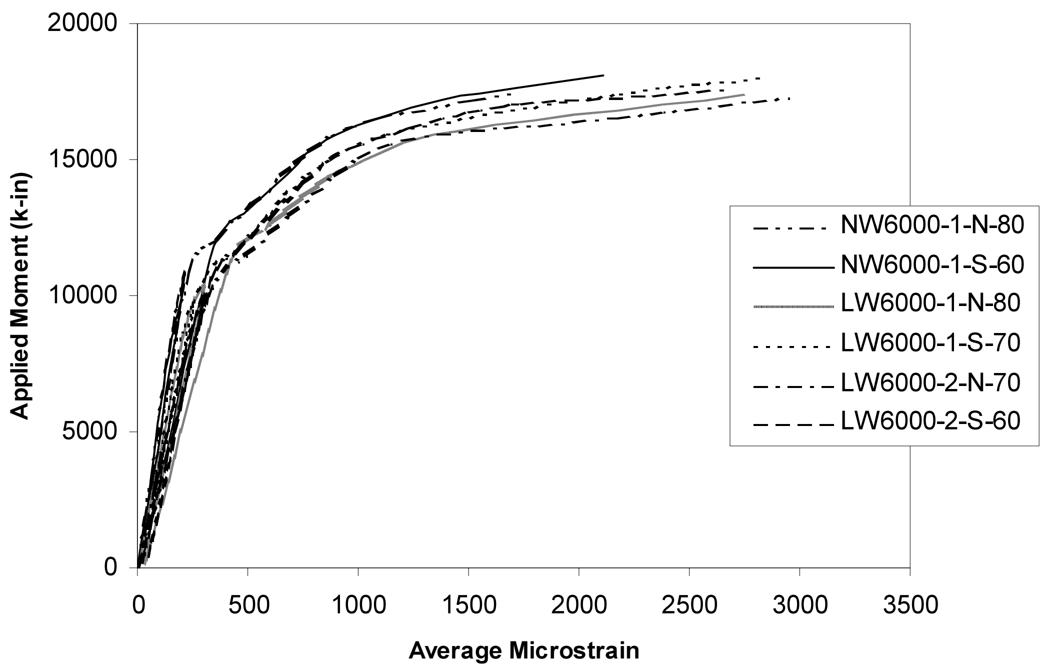

Figure 11.8 Applied Moment vs. Average Microstrain - Kolozs

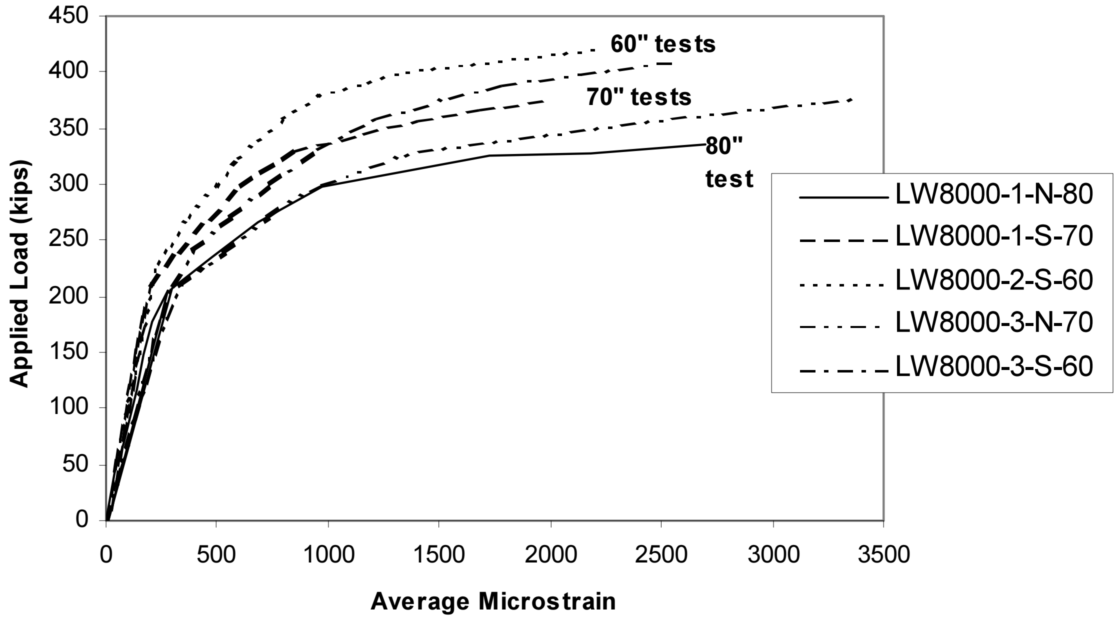

Figure 11.9 Applied Load vs. Average Microstrain - Thatcher

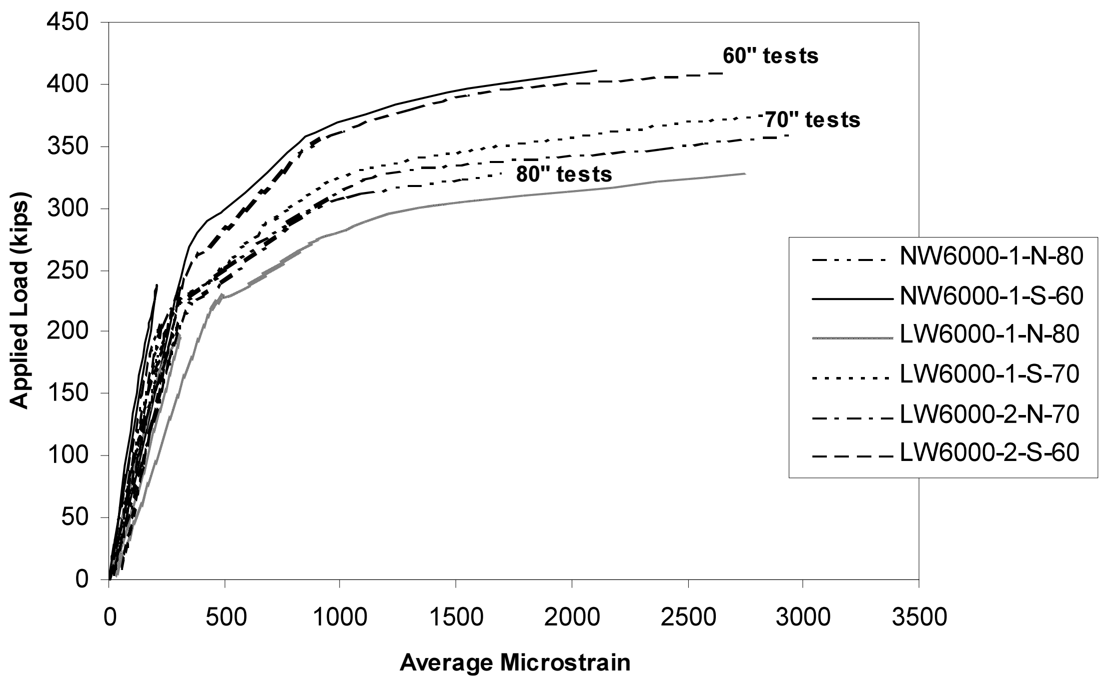

Figure 11.10 Applied Load vs. Average Microstrain - Kolozs

The ultimate deflections are shown in Figure 11.11. The deflections achieved were similar for all tests, since a deflection of approximately 3 in (76.2 mm) was used as one of the criteria for stopping the test procedure. The tests on LW6000-1-S-70 and LW8000-3-N-70 were pushed to a further limit to observe deck crushing and load pickup at high levels of strain. The initial test on NW6000-1-N-80 was stopped earlier due to inexperience and safety concerns, and the test on LW8000-3-S-60 was stopped due to high strains at the top of the composite deck.

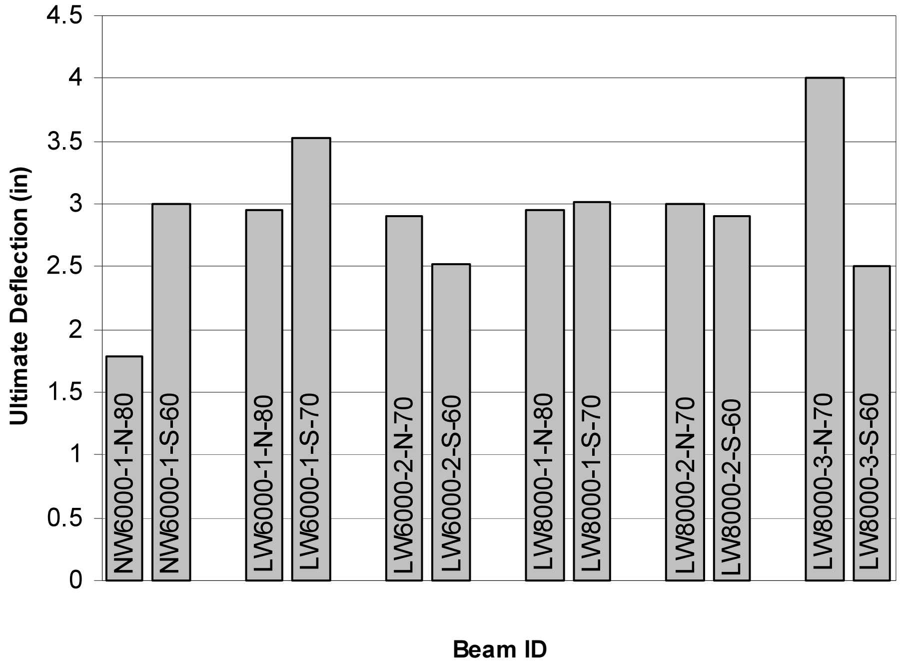

Figure 11.11 Ultimate Deflections

11.2.6 Crack Patterns

The crack patterns developed during each test are presented in this subsection. The results from each individual beam will not be presented here, due to the uniformity of the crack formation in the beams during each of the tests. Rather, this section gives a general presentation of the resulting crack patterns.

The cracks were marked on the beams with colored markers at each load increment until ultimate. One color was used for flexural cracks, while another was used for shear cracks. The crack patterns were divided into three distinct regions, or zones. Zone 1 extended from the end of the beam to the first loading point, a distance equal to the embedment length of the test. This zone was dominated by shear cracks. The shear cracks formed in the web, and extended at approximately 45 degree angles in both directions as the load increased. These cracks formed and then propagated all the way down to the support point of the beam at the bearing pad. Some flexural cracks formed in this zone as well. The flexural cracks formed at the bottom of the beam and extended up at a slight inclination as the load increased. Zone 1 is shown in Figure 11.12.

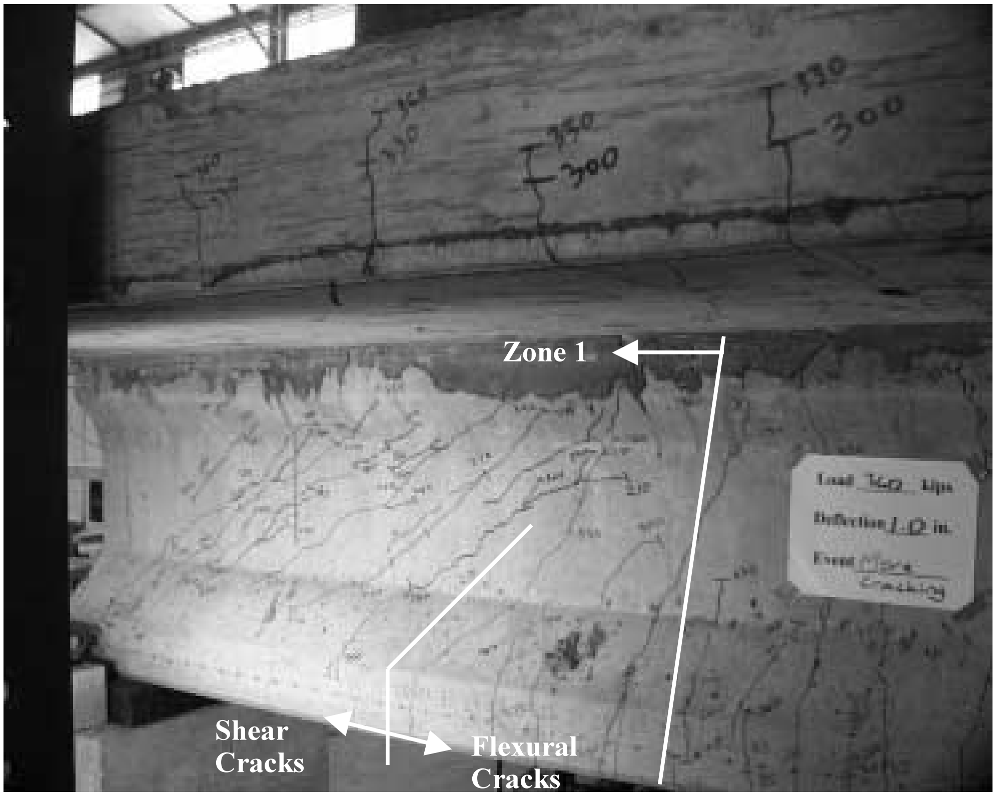

Figure 11.12 Zone 1 Cracking

Zone 2 extended from the end of Zone 1 to the second loading point, a distance equal to 36 in. This zone was dominated by flexural cracks, and was the essentially constant moment region developed beneath the loading points. The flexural cracks formed at the bottom of the beam and extended up until they reached the neutral axis depth at each loading. These cracks lengthened as the load increased, and the number of them increased with load as well. The flexural cracks extended nearly vertically, with those at the edge of the zone showing a slight inclination. Zone 2 cracking is shown in Figure 11.13.

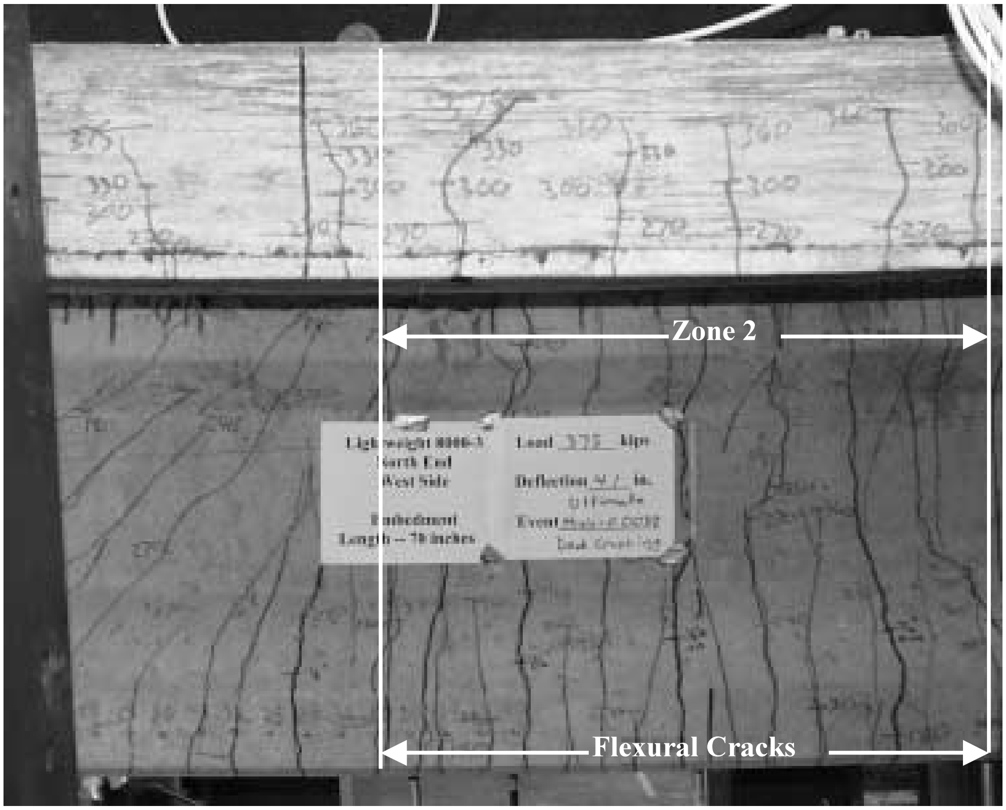

Figure 11.13 Zone 2 Cracking

Zone 3 extended from the end of Zone 2 to the other support point of the beam. Zone 3 was dominated by shear cracks. The shear cracks formed in the web and extended at approximately 45 degree angles in both directions as the load increased. These cracks extended from the bottom of the beam up to the deck of the composite beam and into the deck for some cracks. The shear cracks in this zone did not form gradually, however. During each successive load increment, another crack or two would spring forth with an audible pop, a slight distance further down the beam than the previous one. Some flexural cracks formed in Zone 3 as well, in the area closest to Zone 2. The flexural cracks formed at the bottom of the beam and extended up at a slight inclination as the load increased. Zone 3 is shown in Figure 11.14.

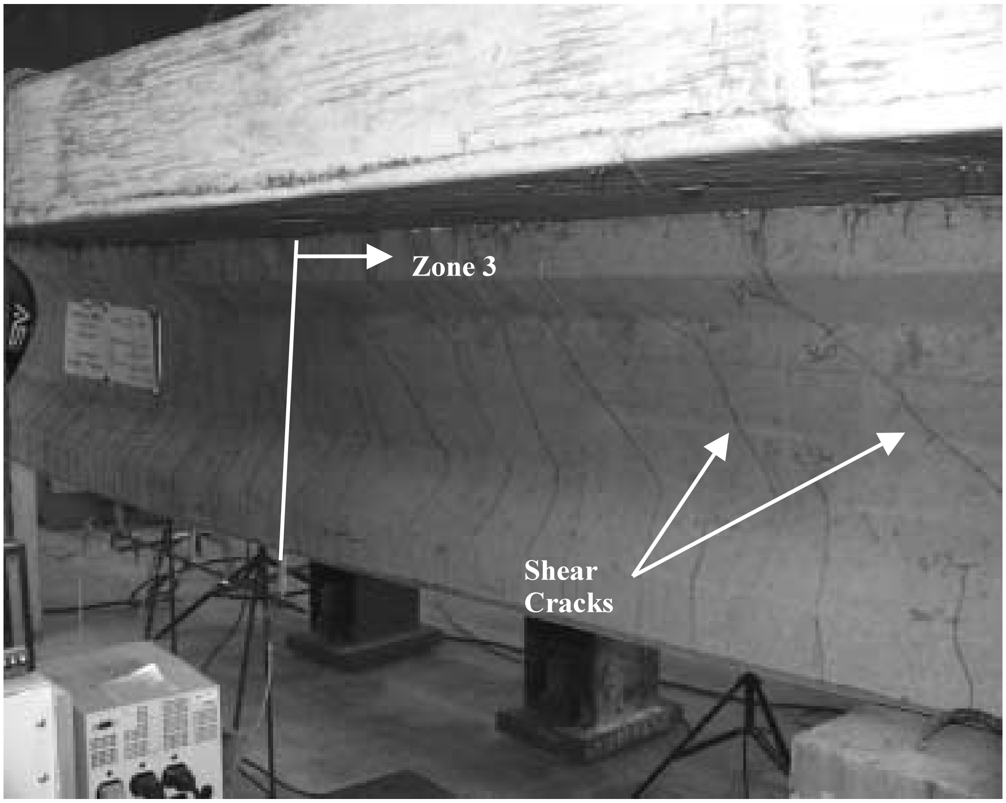

Figure 11.14 Zone 3 Cracking

11.2.7 Strand Slip

A single strand slipped during the testing of LW6000-2-S-60 beam, and was attributed to oil on the strand. No other strand slips occurred. For all of the strands in each of the tests excluding the isolated slip reported above, the measured slip was less than 0.01 in. Slips of this small magnitude can be attributed to electronic noise in the data acquisition equipment. A slip of a strand would have indicated a bond failure, which would have established that the development length required was greater than the embedment length being tested. However, since none of the strands slipped, their full strength was developed. Therefore, the development length is some distance less than the minimum embedment length tested of 60 in. The results are shown in Table 11.5.

Table 11.5 Strand Slip

Beam ID	Embedment Length, in / (mm)	Maximum Strand Slip, in / (mm)
NW6000-1-N-80	80 (2,032)	< 0.01 < (0.25)
NW6000-1-S-60	60 (1,524)	< 0.01 < (0.25)
LW6000-1-N-80	80 (2,032)	0.3 (7.62)
LW6000-1-S-70	70 (1,778)	< 0.01 < (0.25)
LW6000-2-N-70	70 (1,778)	< 0.01 < (0.25)
LW6000-2-S-60	60 (1,524)	< 0.01 < (0.25)
LW8000-1-N-80	80 (2,032)	< 0.01 < (0.25)
LW8000-1-S-70	70 (1,778)	< 0.01 < (0.25)
LW8000-2-N-70	70 (1,778)	< 0.01 (7.62)
LW8000-2-S-60	60 (1,524)	< 0.01 < (0.25)
LW8000-3-N-70	70 (1,778)	< 0.01 < (0.25)
LW8000-3-S-60	60 (1,524)	< 0.01 < (0.25)

11.2.8 Failure Types

Several types of failure were observed during the testing of the 6000 psi and 8000 psi beams, and are presented in this subsection. They are listed in Table 11.6. All of the beams were tested to some form of flexural failure; however, the exact manifestation of the failure varied for each beam and each test. The failure state was limited to a flexural one by the design of the beam, which included heavy shear reinforcement to preclude a shear failure, and by the excellent embedment that fully developed all of the stands in less than L_e = 60 in. The criterion for a flexural failure varied somewhat for each test, but was generally one of high deflection or strain in the beam at ultimate load.

Table 11.6 Types of Failure

Beam ID	Type of Failure
NW6000-1-N-80	Flexural Failure
NW6000-1S-60	Flexural Failure, Deck Crack Initiation
LW6000-1-N-80	Flexural Failure, Deck Crack Initiation
LW6000-1-S-70	Flexural Failure, Strand Slip, Deck Crack
LW6000-2-N-70	Flexural Failure, Complete Deck Crack, V-Crack, Spalling at Support in Beam Flange
LW6000-2-S-60	Flexural Failure, Deck Cracking Initiation, V-Crack, Spalling at Support in Beam Flange
LW8000-1-N-80	Flexural Failure
LW8000-1-S-70	Flexural Failure
LW8000-2-N-70	Flexural Failure, V-Crack, Spalling at Support in Beam Flange
LW8000-2-S-60	Flexural Failure, Complete Deck Crack, V-Crack, Spalling at Support in Beam Flange
LW8000-3-N-70	Flexural Failure, Deck Cracking Initiation
LW8000-3-S-60	Flexural Failure, Complete Deck Crack, Spalling at Support in Beam Flange

The modes of failure in the 6000 psi beams were similar to those of the 8000 psi beams, as Table 11.6 shows. To condense the discussion, only the 8000 psi beam series will be discussed in the remainder of this subsection.

The first 8000 psi beam tested was constructed with a normalweight concrete deck. The two tests performed on this beam resulted in typical flexural failures, with large strains and deflections. The second 8000 psi beam tested was constructed with lightweight precast deck panels and normalweight concrete for the remainder of the deck. The two tests performed on this beam exhibited flexural failure characteristics like the first beam, with the addition of two other types of secondary failure. A crack formed in the deck at a level equal to the top of the lightweight panels in the deck that, instead of continuing vertically up the deck, veered off to either side in a V-shape, as seen in Figure 11.15. A type of hinging action at that point in the deck likely caused this crack formation. No steel was placed to interconnect the panels. Thus, when the load on the deck began to cause significant deflection, the panels began to rotate with respect to one another. The crack then tended to spread laterally as the beam deflected.

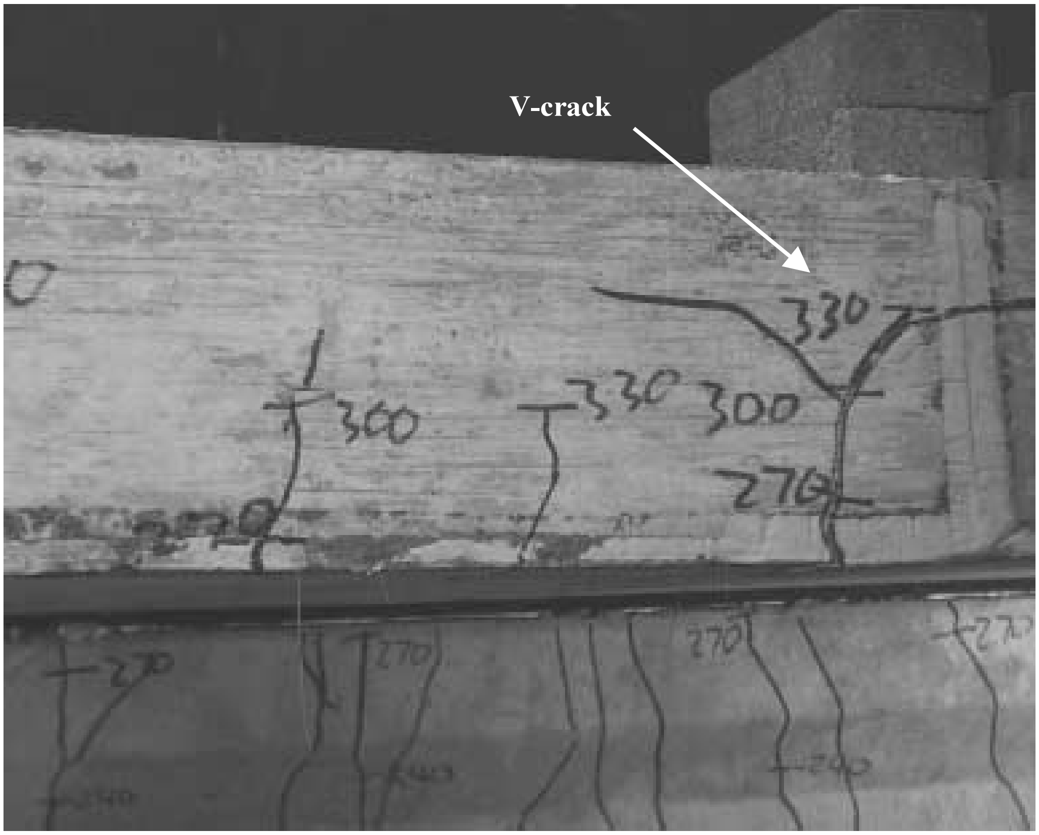

Figure 11.15 V-crack in Deck

Another form of failure that occurred in several of the lightweight concrete beam tests was a spalling of the concrete that occurred on one side at the end of the beam at loads near ultimate. This spalling will be discussed in the following subsection in more detail.

The final three 8000 psi tests were pushed a bit further than the previous three, due to confidence in obtaining meaningful data while maintaining safety during the testing, as experience increased. These tests achieved cracking in the deck due to compressive stresses. Two of the 8000 psi tests reached a point where deck crushing occurred, as seen in Figure 11.16.

11.2.9   Support Spalling

Five of the twelve tests had support spalling prior to ultimate. This condition was not considered to be an ultimate condition, since the ultimate load of the beam was not affected by the formation of the spall. The spalls were very similar in appearance in all five tests, and occurred on only one side of the beam in each case. They are shown in Figures 11.17 to 11.19.

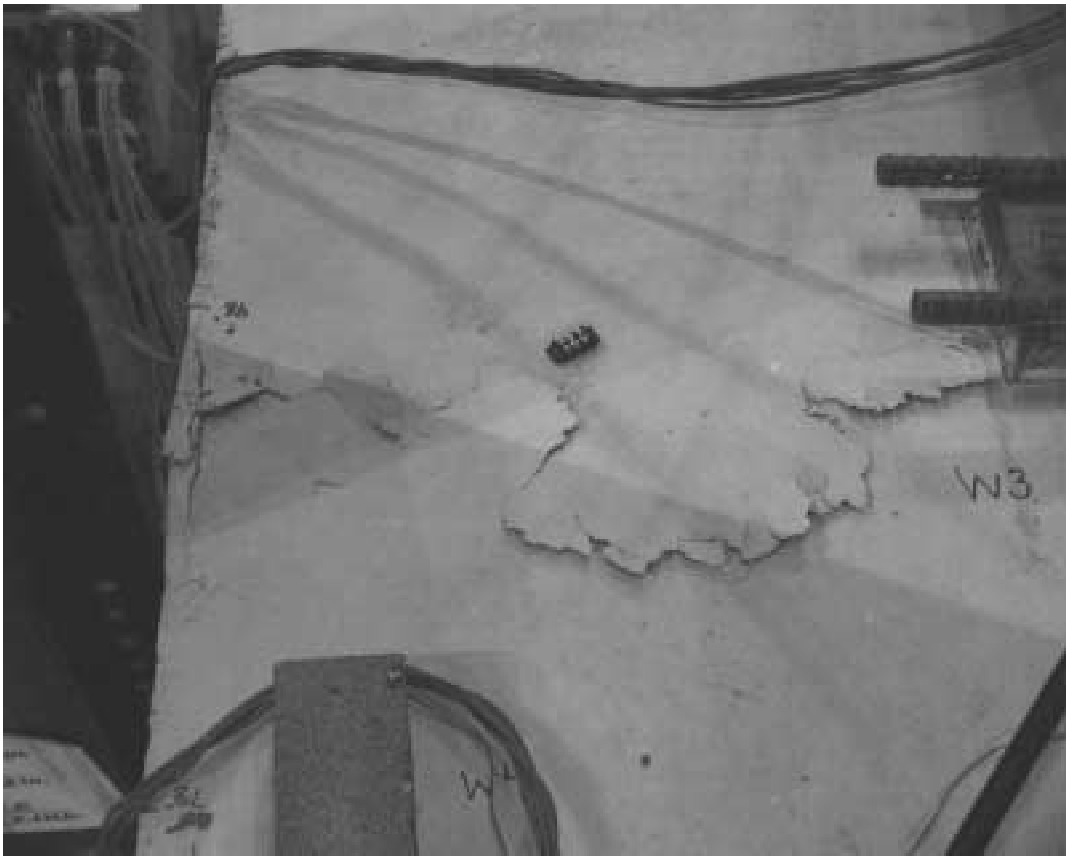

Figure 11.16 Deck Crushing

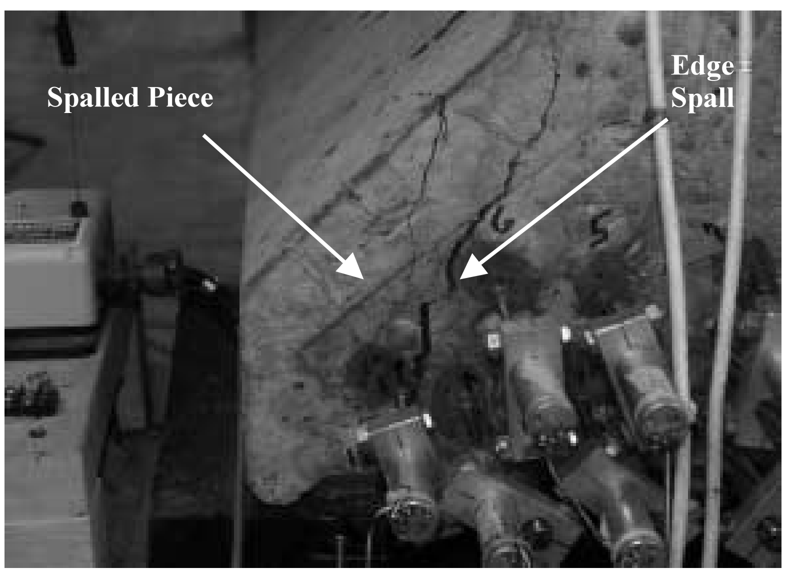

Figure 11.17 Support Spall at End (LW8000-2-N-70)

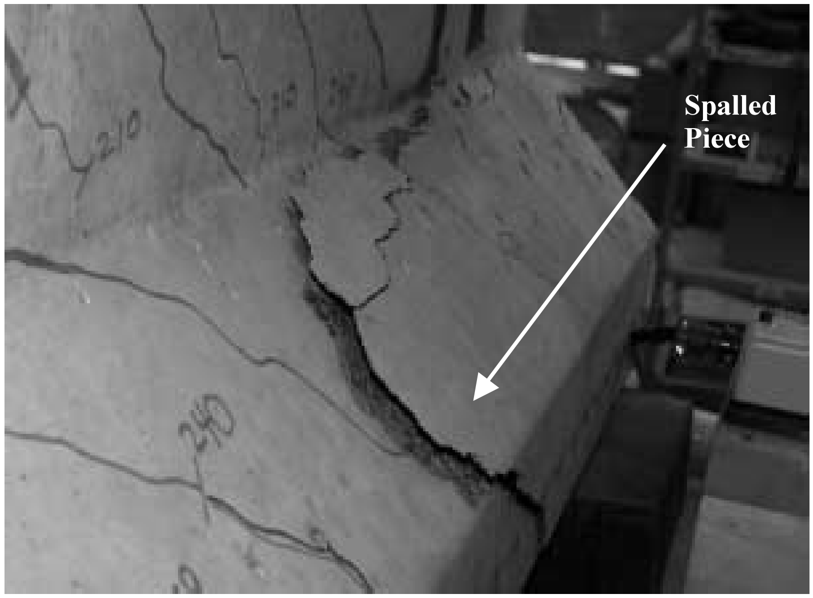

Figure 11.18 Support Spalling (LW8000-2-N-70)

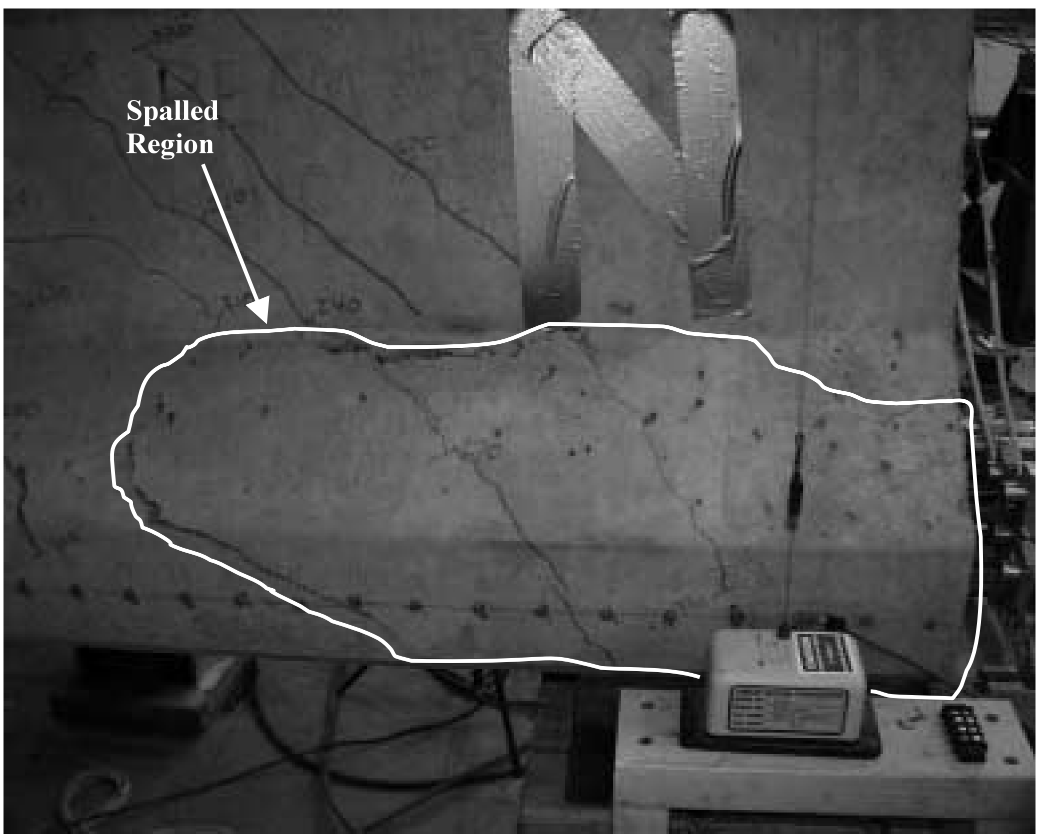

Figure 11.19 Support Spalling Area (LW8000-2-N-70)

The load at which the spall occurred is shown in Figure 11.20 as a ratio of the spalling load relative to the cracking load. The first time the spall occurred, while testing LW6000-2-N-70, the test was stopped. On future tests, the loading was continued after the spall was noted and measured. For LW6000-2-S-60, the spall occurred at 97% of the ultimate load during the test. For LW8000-2-N-70 and LW8000-3-S-60, the spalls occurred at 92% and 98%, respectively, of the ultimate load during the test. For LW8000-2-S-60, the spall occurred at 75% of the ultimate load during the test, but the ultimate load was not affected by the formation of the spall. The occurrence of the spall in each case was at a load level more than twice the service load level if dead load is taken equal to live load, and more than three times the service load level if dead is taken equal to twice live, which is more common for bridge applications. This indicates that the spalling is not a service level occurrence, but rather an ultimate level occurrence. An explanation of the formation of the support spalls is discussed in the Section 11.3.2.

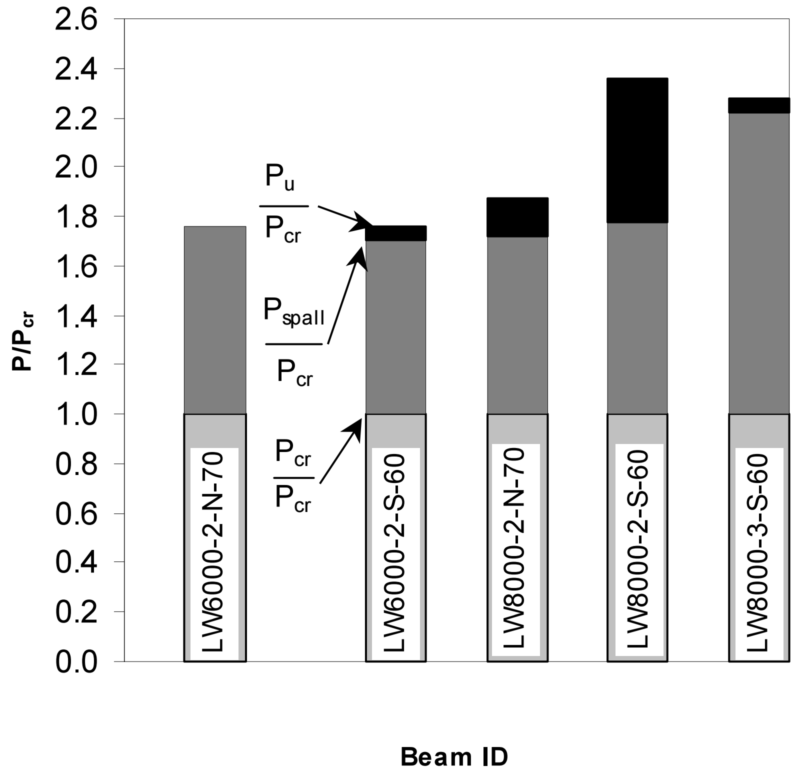

Figure 11.20 Support Spalling Loads

11.3 DISCUSSION

This section presents a discussion of the results of the 6000 psi and 8000 psi beam tests. The test results are discussed, a theory for the formation of the support spalls is introduced, and behavior observed with the use of the lightweight panels is discussed.

11.3.1 Test Results

This subsection compares the results obtained from the testing of the 6000 psi and 8000 psi beams.

11.3.1.1 Moment Comparison

Three methods of calculating the theoretical moment capacity were used: the AASHTO STANDARD Method, the Whitney Stress Block method, and the Stress Block Factors Method [24]. A factor of $\phi = 1.0$

was used in each method. All of these methods are based on the assumption of strain compatibility and give virtually identical results for these beams. The results of these calculations are shown in Table 11.7, along with the ultimate moment achieved by the beams during testing. This moment includes both the applied moment and the dead load moment at the critical section. The ultimate-to-calculated moment ratios are shown as well, and demonstrate reasonable correlation between test groups. The three calculation procedures gave almost identical results.

Table 11.7 Moment Comparison

Beam ID	AASHTO Standard, k-in	Whitney Stress Block, k-in	Stress Block Factors, k-in	Ultimate Applied Moment, k-in	Ultimate/ Calculated
NW6000-1-N-80	14,788	14,850	14,870	17,438	1.17
NW6000-1-S-60	14,829	14,890	14,900	18,125	1.21
LW6000-1-N-80	14,773	14,830	14,850	17,437	1.18
LW6000-1-S-70	14,773	14,830	14,850	17,974	1.21
LW6000-2-N-70	14,654	14,860	14,870	17,254	1.16
LW6000-2-S-60	14,654	14,860	14,870	17,590	1.18
Averages	**14,745**	**14,853**	**14,868**	**17,636**	**1.18**
LW8000-1-N-80	14,743	14,800	14,820	17,908	1.21
LW8000-1-S-70	14,743	14,800	14,820	17,974	1.21
LW8000-2-N-70	14,758	14,810	14,830	17,684	1.19
LW8000-2-S-60	14,758	14,810	14,830	18,310	1.24
LW8000-3-N-70	14,964	15,050	15,060	18,128	1.20
LW8000-3-S-60	14,964	15,050	15,060	17,593	1.17
Averages	**14,822**	**14,887**	**14,903**	**17,933**	**1.20**

The ultimate moment achieved by the beams exceeded the theoretical calculated moment in the 6000 psi tests by an average of 18%, and in the 8000 psi tests by an average of 20%. These differences are partially due to the generally conservative nature of the design theory and probably more to a higher ultimate strand stress than the reported 270 ksi. Previous testing of pretensioned beams at FSEL indicates that the ultimate stress of the strand is usually 278-280 ksi rather than 270 ksi reported in the mill tests. The difference between the two groups of tests is probably due to the increased experience of Thatcher who carried out the tests to a higher level of loading and often to full crushing at the deck rather than discontinuing loading at very large deflections.

11.3.1.2 Displacement Comparison

This subsection compares the deflections of the beams during all twelve tests. The plots of applied moment versus deflection are shown in Figures 11.22 and 11.23, with all six tests plotted together in order to show direct comparisons. Data from Thatcher's testing is presented in Figure 11.22 [75], and data from the testing carried out by Kolozs is presented in Figure 11.23 [43]. The figures show that the deflection for all of the tests was very similar, except for NW6000-1-N-80, which was stopped early for safety concerns due to being the first test, and LW8000-3-N-70, which was pushed further in order to observe additional crushing behavior in the deck.

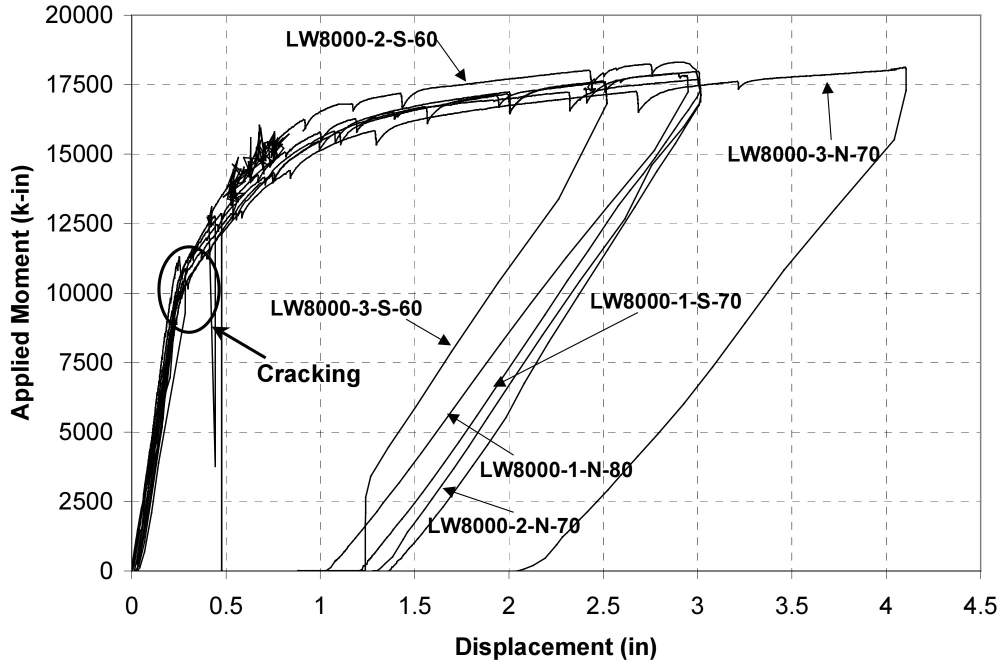

Figure 11.21 Moment-Displacement Comparison – Thatcher

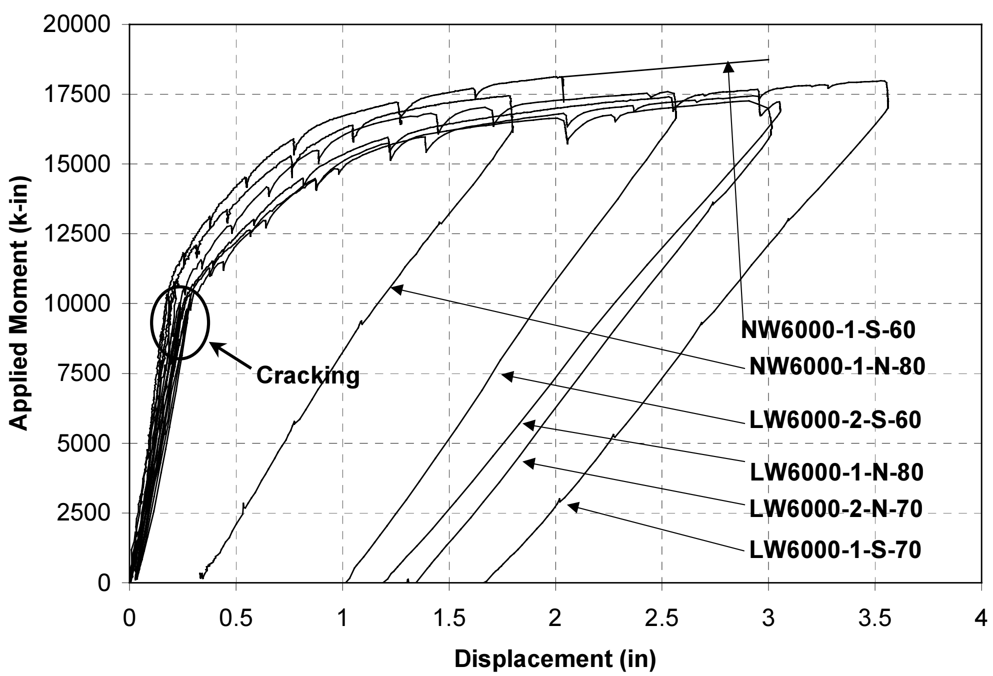

Figure 11.22 Moment-Displacement Comparison – Kolozs

11.3.1.3 Development Length Comparison

This subsection compares the tested embedment lengths with the calculated development lengths of the beams. The development lengths were calculated using several different methods. The results from these calculations are shown in Table 11.8, as are the results of the development length testing. In all twelve tests, the calculated development length exceeded the actual embedment length of the beam. This was verified by testing each beam at a specified embedment length. The equation proposed by Mitchell [54] came the closest to the actual range of the development length, which was experimentally determined to be less than 60 in (1524 mm). It should be noted that all of the methods shown were developed for normalweight concrete. The test results show that the equations are also valid for lightweight concrete. The Mitchell equation may come the closest, because it takes into account the square root of the compressive strength of the concrete. This has some reflection of the variance in the modulus of elasticity of the concrete, since the analytical expression for the modulus of elasticity uses the square root of the concrete compressive strength multiplied by a constant that does however differ with the unit weight of the concrete. In any case, these tests indicate that the actual development length is substantially less than the widely used expressions and the AASHTO design values for both normalweight and lightweight concrete beams.

Table 11.8 Development Length Comparison

Author	Development Length Equation	L_d NW6000 in	L_d LW6000 in	L_d LW8000 in
ACI 318 / AASHTO [2,1]	$L_d = \left(f_{ps} - \frac{2}{3} f_{se} \right)$	82	86	86
Zia & Mostafa [18]	$L_d = L_t + 1.25(f_{pu} - f_{se})d_b$	88	92	92
Buckner (FHWA) [5]	$L_d = L_t + \lambda(f_{ps} - f_{se})d_b$	78	104	102
Mitchell [13]	$L_d = L_t + (f_{ps} - f_{se})d_b \sqrt{\frac{4.5}{f'_c}}$	69	67	68
Actual L_d	Determined from testing	< 60	< 60	< 60

11.3.2 Support Spalling

This subsection discusses the support spalling that occurred during five of the lightweight concrete beam tests. The detailed explanation for the formation of the support spalls is a topic for further study. No clear correlation between these local failure occurrences was found, and the ultimate load for the beam was not affected. No similar spalls occurred in the normalweight concrete beam tests. No spalls occurred during the four tests of the lightweight beams with full depth normalweight concrete decks. Spalls did occur during all four tests on lightweight concrete beams with both lightweight concrete deck panels and with normalweight concrete topping decks. However, only one spall occurred during the two tests on the lightweight concrete beam with the full-depth lightweight deck. Therefore, no direct correlation between lightweight concrete decks and the formation of the spalls can be drawn, although they appear to prevail when lightweight concrete panels are used with lightweight concrete beams. Kolozs presented a possible explanation for the formation of the spalls:

> “Due to the lower tensile strength of the lightweight concrete, it was theorized that the distributed load applied by the bearing pad put the flange of the beam into deep beam bending. Also, due to the curvature of the beam, the beam had to bear on less area as the load increased and therefore more stress was concentrated in a smaller area of the beam flange. The failure looked very similar to a split cylinder test. Once the stress exceeded the rupture stress, a crack was initiated and as the stress redistributed, it exceeded the cracking strength of the concrete and the crack unzipped along the length of the concrete in the flange of the support.” [43]

If this theory were completely true, the spalling would have been expected to occur on all of the lightweight beam tests. Since this was not the case, an additional modification to the theory was presented by Thatcher.

> “It was possible that some longitudinal rotation, or torsion, developed as the beam deflected such large amounts. Whether it was due to an initial eccentricity in the load or merely due to the deflection of the beam is not critical. Due to this rotation, the bearing area of the end of the beam was decreased even more and the tensile stresses increased beyond the capacity of the lightweight concrete. This would account for the spalling that occurred in some of the tests and not in others, and always on only one side of the beam. Additionally, local imperfections in the bottom of the beam “as built” could account for the formation of the spalls. Again, further study of this matter is required before a definitive answer can be provided, but the ultimate capacity of the composite beam was not affected by the local concrete spalling at the support.” [75]

No direct correlation between lightweight concrete decks and the formation of the spalls could be drawn. Examining the results from all of the tests did not provide any additional observations beyond those already discussed. Kolozs found no spalls in the testing of the normalweight beam, and this would be consistent with the theory presented by Thatcher. To repeat, “further study of this matter is required before a definitive answer can be provided, but the ultimate capacity of the composite beam was not affected by the local concrete spalling at the support.” [75]

11.3.3 Lightweight Deck Panels

This section discusses the use of the lightweight deck panels during this project. The panels presented no difficulty when placing them in the formwork. On a construction site, the panels will negate the need for such formwork. However, since only one beam was being tested at a time, formwork was needed in the study to support the panels’ outside edges. The panels fit together well, and were easily managed by an overhead crane and one set of hands to direct the final placement.

During the testing, the panels behaved very well. Although the initial stiffness of the composite beam was lower, this was due to the use of lightweight concrete in the panels, and not the presence of the panels themselves. The full depth lightweight concrete deck exhibited an even lower initial stiffness than the deck with panels. The panels introduced a new form of crack propagation with the V-cracks discussed previously and shown in Figure 11.15. However, these cracks did not affect the ultimate strength of the beam. The spalling at the support occurred during both tests of the beam with the lightweight panels; however, the panels are not believed to be the cause of the spalling, as discussed previously

Use of the panels during the 6000 psi and 8000 psi beam tests produced similar results in terms of ease of use and performance. Overall, the panels were found to be both useful and behaviorally satisfactory.

CHAPTER 12: CONCLUSIONS FOR THE BEAM PORTION OF THE PROJECT

12.1 INTRODUCTION

Conclusions from the mix design portion of this project are given in Chapter 6. This chapter covers the summary and conclusions derived from the nominal 6000 psi and 8000 psi concrete strength beam test results. A summary of the results is presented, followed by conclusions drawn. Lastly, recommendations are made.

12.2 SUMMARY

The objective of Project 0-1852 is to examine the feasibility of using high performance lightweight concrete in prestressed bridge girders in Texas. The use of prestressed lightweight concrete panels as bridge deck formwork was also included in the project. This report draws conclusions from the results of the testing process.

The manufacture of the test beams was discussed in Section 2.2. The beams were produced at Heldenfel's Prestressing Plant in San Marcos, Texas. The beams were precast pretensioned lightweight concrete with 12 – ½ in (12.7 mm) diameter, Grade 270 ksi (1,861 MPa), low-relaxation steel strands. After the forms were removed and the transfer length instrumentation was in place, the beams were brought to Ferguson Structural Engineering Laboratory of the University of Texas at Austin for testing. The setup of the testing program was outlined in Section 2.3, and the actual test procedure was presented in Section 2.4.

The project's four main aspects were the lightweight concrete, the transfer length of the beams, the development length of the beams, and the use of prestressed lightweight concrete panels. These four aspects are examined in greater detail in the follow subsections.

12.2.1 Lightweight Concrete

The concrete mixes used in the manufacture of the test beams were developed by Heffington and Kolozs, and reported by Heffington [33]. The normalweight concrete mix was designed to be of nominal 6000 psi compressive strength. The lightweight concrete mixes were designed to be of nominal 6000 psi compressive strength and nominal 8000 psi compressive strength, respectively. The unit weight of the resulting lightweight concrete from the mix was required to fall between 118-122 lb/ft^3. The properties of the mixes are shown in Table 12.1.

Table 12.1 Concrete Properties

Beam ID	Compressive Strength (f 'c), psi		Unit Weight lb/ft^3
	1-day	Long Term	
NW6000	3,490	5,500	149
LW6000	4,900	8,130	118
LW8000	5,560	7,850	122

The manufacture of the beams with lightweight concrete took place without special consideration by the plant workers. No significant problems were experienced that might not come about from the use of normalweight concrete.

12.2.2 Transfer Length

Transfer length is defined as the distance required to transfer the fully effective prestressing force from the strand to the concrete in a pretensioned prestressed concrete beam. The determination of this quantity was accomplished by two different methods: measurement of the concrete strains and strand draw-in.

The concrete strain measurements were performed using a DEMEC Strain Measurement System. The strain was measured every 50 mm using a 200-mm gauge length. The data was reduced to give a strain profile for each end of the beams. The transfer length was then determined for each profile using the 95% Average Maximum Strain Method [65]. Due to variability in the data of the individual strain profiles, the data from similar beams were averaged to obtain average strain profiles for which the transfer length was determined.

Unlike the strain measurements, the draw-in testing data could not be used directly to give the transfer length. The transfer length was determined by applying the draw-in data with Equation 4.3, assuming α=2.

A summary of the average transfer lengths results from the individual strain profiles, average strain profiles and draw-in values are given in Table 6.2.

Table 12.2 Summary of Transfer Length Results

Beam ID	Transfer Strength (L_t), in		
	Individual Strain Profiles	Average Strain Profiles	Strand Draw-In
NW6000	18.3	18.2	15.6
LW6000	30.9	35.8	15.3
LW8000	35.3	34.4	13.1

12.2.3 Development Length

Testing the beams at a certain embedment length and observing the type of failure that occurred allowed the development length of the beams under consideration to be approximated. A flexural failure with no indication of bond failure at a particular embedment length indicated that the development length of the beam was less than that tested. A range of embedment length tests, from 80 in down to 60 in, was performed in order to isolate the approximate development length of the beams.

The beams were instrumented in order to record various types of behavior during the testing process. Concrete strain, horizontal and vertical deflections at the supports, vertical deflections of the beam at the load points, strand slip, and applied load were all measured and recorded. The clearest measurement of bond failure would be from the potentiometers used to measure strand slip. Because the number of beam tests performed was limited the fact that no bond failures occurred at the shortest embedment length of 60 in indicates the actual development length was some value less than 60 in.

The embedment length, maximum strand slip, development length upper bound, as well as the ultimate moment developed in the beams, are collectively reported in Table 12.3.

Table 12.3 Development Length Summary

Beam ID	Embedment Length	Maximum Strand Slip, in	Development Length Upper Bound, in	Ultimate Moment, k-in	Actual/ Predicted Ultimate Moment
NW6000-1-N-80	80	<0.01	<60	17,908	1.17
NW6000-1-S-60	60	<0.01	<60	17,974	1.21
LW6000-1-N-80	80	0.3	<60	17,684	1.18
LW6000-1-S-70	70	<0.01	<60	18,310	1.21
LW6000-2-N-70	70	<0.01	<60	18,128	1.16
LW6000-2-S-60	60	<0.01	<60	17,597	1.18
LW8000-1-N-80	80	<0.01	<60	17,908	1.21
LW8000-1-S-70	70	<0.01	<60	17,974	1.21
LW8000-2-N-70	70	<0.01	<60	17,684	1.19
LW8000-2-S-60	60	<0.01	<60	18,310	1.24
LW8000-3-N-70	70	<0.01	<60	18,128	1.20
LW8000-3-S-60	60	<0.01	<60	17,593	1.17

Average, All Tests	**1.19**
Standard Deviation, All Tests	**0.024**

12.2.4 Lightweight Panels

Prestressed lightweight concrete panels were added to the scope of the project. The panels are used as formwork for the deck, and comprise half of the depth of the deck. The panels were manufactured and placed without any difficulty, as discussed in Chapters 8 and 10.

The behavior of the panels during testing was discussed in Section 11.3.3. The use of the panels introduced a new form of crack propagation. V-cracks formed in the deck at the joints between the panels, but these cracks only occurred near failure of the composite beam and deck crushing. The support spalls that formed during testing of the beams with panels are not directly linked to panel use, and occurred only at loads near ultimate.

12.3 CONCLUSIONS

This section presents the conclusions drawn from the test results of the nominal 6000 psi and 8000 psi beam tests.

12.3.1 Transfer Length

The following conclusions can be drawn about the transfer length of normalweight and lightweight concrete based on the tests performed in this research study:

1) The ACI and AASHTO code expressions are a conservative estimate of the transfer length of normalweight concrete, but underestimate the transfer length of lightweight concrete.

2) The transfer length at transfer of the prestressing force for ½-in strands in the normalweight concrete beams in this study was 18.3 in (37 d_b).
3) The transfer length at transfer of the prestressing force for ½-in strands in the lightweight concrete beams in this study was 35.1 in (70 d_b).
4) The transfer length at transfer of the prestressing force for 3/8-in. strands in the lightweight concrete deck panels in this study was 16.1 in (43 d_b). This was about 10% longer than for normalweight concrete panels.
5) Most of the models developed to predict transfer length were developed for normalweight concrete and do not accurately model the behavior of lightweight concrete.
6) The modulus of elasticity is an important factor in determining the transfer length for both normalweight and lightweight concrete.
7) Buckner's model for determining transfer length, though developed for normalweight concrete, is a conservative and accurate estimate for both normalweight and lightweight concrete.

12.3.2 Development Length

The following are conclusions regarding the development length of the normalweight and lightweight concrete test beams:

1) The AASHTO/ACI development length equation is conservative for normalweight concrete.
2) The AASHTO/ACI development length equation is conservative for lightweight concrete.
3) The development length of the ½ in strand in the normalweight concrete beam in this study is less than 60 in.
4) The development length of the ½ in strand in the lightweight concrete beams in this study is less than 60 in.
5) The use of the prestressed lightweight concrete panels was simple and straightforward during the construction process.
6) The use of the prestressed lightweight concrete panels had no significant adverse effect on the behavior of the composite beams, and did not affect the development length of the ½ in strand in the beams.
7) The localized support spalling that occurred near ultimate in the lightweight concrete beams was not a critical failure condition.

12.3.3 Ultimate Moment Capacity

The ultimate moment capacity developed by the lightweight concrete girders and the load-deflection curves for those girders were virtually identical to the normalweight concrete girders. The full AASHTO ultimate moment capacity of the girders was developed with a substantial reserve as shown in Table 12.3.

12.3.4 Recommendations

The following are recommendations for the AASHTO Bridge Design Specification and for application of the results of this study:

1) The development length models identified in this report and by Kolozs [43] and Thatcher [75] all appear to be conservative for both normalweight and lightweight concrete. AASHTO should consider modifying its requirements to reflect this fact.

2) If an economic analysis shows feasibility, lightweight concrete can be used in the design and construction of bridge girders in Texas. Such use could result in significant savings in the dead load of the structure.

3) If a check of service load level tensile stresses using the reduced split tensile strength of lightweight concrete and an economic analysis shows feasibility, prestressed lightweight concrete deck panels can be used in the design and construction of bridge decks in Texas.

12.3.5 Future Study

The following areas are recommended for future study and clarification:

1) Determination of the exact development length of the ½ in strand in the normalweight and lightweight concrete test beams and comparison to design equations.

2) Determination of the exact cause of the support spalling failure that occurred during the testing procedure.

3) Study of the economic benefits of using lightweight concrete in bridge girders.

4) Study of the economic benefits of using prestressed lightweight concrete panels in bridge deck construction.

Appendix A: Notation

a	Distance from first load point to location of actuator
b	Coefficient indicating diameter of prestressing strand
d_b	Nominal diameter of prestressing strand
E_c	Modulus of elasticity of concrete
E_{ci}	Modulus of elasticity of concrete at transfer
f'_c	Concrete compressive strength
f'_{ci}	Concrete compressive strength at transfer
f_{pi}	Initial stress in strands, before relaxation losses
f_{ps}	Stress in prestressing strands at nominal strength
f_{pu}	Ultimate tensile strength of prestressing strands
f_r	Modulus of rupture of concrete
f_s	Stress in prestressing strands
f_{se}	Effective stress in prestressing strands after losses
f_{si}	Stress in prestressing strands immediately before transfer
L_d	Development length of prestressing strands
L_e	Embedment length of prestressing strands
L_{fb}	Flexural bond length of prestressing strand
Li	Distance from face of concrete to measurement point on strand before transfer
L_o	Distance to centerline of support bearing pad from end of beam
L_r	Distance from face of concrete to measurement point on strand after transfer
L_t	Transfer length of prestressing strand
M_u	Ultimate moment strength
P_{cr}	Applied load at first flexural crack
P_u	Applied load at ultimate moment strength
α	Coefficient indicating shape of bond stress distribution in transfer zone

β_1 Factor defining average size of rectangular stress block

ε_{ps} Strain in prestressing strand at nominal strength

ε_s Strain in prestressing strands

ε_{si} Strain in prestressing strand immediately before transfer

Δ_d Change in length of prestressing strand associated with strand draw-in

Δ_e Change in length of prestressing strand associated with elastic shortening

Δ_t Total measured draw-in of prestressing strand

λ Coefficient indicating bond stress distribution

ρ Ratio of tension reinforcement

ρ_b Reinforcement ratio producing balanced strain conditions

ρ_p Ratio of prestressed reinforcement

ω_p $\rho_p f_{ps}/f'_c$

Appendix B: Mix Designs

Table B.1 Mix Designs for First Iteration

(All quantities are pounds per cubic yard unless otherwise noted)

	Cement	Fly Ash	Lightweight Aggregate	Sand	Water
Mix #1	600	0	1155	1387	210
Mix #2	450	150	1155	1371	210
Mix #3	600	0	1260	1207	210
Mix #4	450	150	1260	1181	210
Mix #5	600	0	1155	1122	210
Mix #6	600	0	1271	1013	210
Mix #7	600	0	1328	1090	210
Mix #8	600	0	1155	1122	210
Mix #H-1	800	0	1155	1100	256
Mix #H-2	600	200	1155	1065	256
Mix #H-3	800	0	1260	920	256
Mix #H-4	600	200	1260	885	256

	Daratard	ADVA Superflow	Daravair
Mix #1	6 oz.	30 oz.	0 oz.
Mix #2	6 oz.	30 oz.	0 oz.
Mix #3	6 oz.	30 oz.	0 oz.
Mix #4	6 oz.	30 oz.	0 oz.
Mix #5	6 oz.	30 oz.	1 oz.
Mix #6	6 oz.	30 oz.	0.5 oz.
Mix #7	6 oz.	30 oz.	0 oz.
Mix #8	6 oz.	30 oz.	0.5 oz.
Mix #H-1	8 oz.	40 oz.	0 oz.
Mix #H-2	8 oz.	40 oz.	0 oz.
Mix #H-3	8 oz.	40 oz.	0 oz.
Mix #H-4	8 oz.	40 oz.	0 oz.

Table B.2 Mix Proportions for Second Iteration

(All quantities are in pounds per cubic yard unless otherwise noted)

	Cement	Fly Ash	Lightweight Aggregate	Sand	Water
Mix #1-C	600	200	1161	1098	224
Mix #1-S	600	200	1173	1067	200
Mix #1-W	600	200	1161	1055	224
Mix #2-C	637.5	212.5	1205	983	221
Mix #2-S	637.5	212.5	1205	964	221
Mix #2-W	637.5	212.5	1205	964	221
Mix #3-C	675	225	1103	1003	234
Mix #3-S	675	225	1103	1003	234
Mix #3-W	675	225	1103	1003	234
Mix #4-C	600	200	1231	985	224
Mix #4-S	600	200	1231	985	224
Mix #4-W	600	200	1231	985	224
Mix #5-C	600	200	1184	1031	224
Mix #5-S	600	200	1184	1031	224

	Daratard	ADVA Superflow	Daravair
Mix #1-C	4 oz.	27 oz.	0 oz.
Mix #1-S	4 oz.	39 oz.	0 oz.
Mix #1-W	4 oz.	31 oz.	0 oz.
Mix #2-C	4 oz.	31 oz.	0 oz.
Mix #2-S	4 oz.	39 oz.	0 oz.
Mix #2-W	4 oz.	31 oz.	0 oz.
Mix #3-C	4 oz.	42 oz.	0 oz.
Mix #3-S	4 oz.	39 oz.	0 oz.
Mix #3-W	4 oz.	42 oz.	0 oz.
Mix #4-C	4 oz.	54 oz.	0 oz.
Mix #4-S	4 oz.	46 oz.	0 oz.
Mix #4-W	4 oz.	46 oz.	0 oz.
Mix #5-C	4 oz.	31 oz.	0 oz.
Mix #5-S	4 oz.	39 oz.	0 oz.

Table B.3 Mix Proportions for Third Iteration

(All proportions are pounds per cubic yard unless otherwise noted)

	Cement	Fly Ash	Lightweight Aggregate	Sand	Water
Mix #F-1	412.5	137.5	1333	1159	198
Mix #F-2	450	150	1244	1186	210
Mix #F-3	450	150	1300	1130	210
Mix #F-4	450	150	1300	1130	210
Mix #F-5	494	165	1239	1126	217
Mix #F-6	600	200	1231	985	224
Mix #F-7	671	316	1123	1029	247
Mix #F-8	671	316	1123	1029	247
Mix #F-9	671	316	1153	978	247

	Daratard	ADVA Superflow	Daravair
Mix #F-1	0 oz.	72 oz.	0 oz.
Mix #F-2	0 oz.	33 oz.	0 oz.
Mix #F-3	4 oz.	33 oz.	0 oz.
Mix #F-4	5 oz.	43 oz.	0 oz.
Mix #F-5	12 oz.	34 oz.	0 oz.
Mix #F-6	5 oz.	43 oz.	0 oz.
Mix #F-7	12 oz.	54 oz.	0 oz.
Mix #F-8	27 oz.	54 oz.	0 oz.
Mix #F-9	16 oz.	54 oz.	0 oz.

Appendix C: Test Results for Mixes

Table C.1 Results for Mix Designs in Initial Iteration

	6000 psi Mixes							
Mix Number	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Compressive Strength (psi)								
1 Day	3227	3273	3890	3954	5364	4714	4197	3193
3 Day	3636	3265	4642	5442	6318	6085	5730	4424
7 Day	3827	3959	5007	5713	6793	6363	6277	5033
28 Day	4794	4921	4042	6841	7039	7498	6789	5667
Modulus of Elasticity (ksi)								
1 Day	3174	1815	2111	2177	2389	2224	2147	1938
28 Day	3591	3506	3460	2918	2816	2648	2701	2377
Modulus of Rupture (psi)								
1 Day	n/a	503	508	525	540	525	488	418
28 Day	746	724	765	725	765	810	761	670
Splitting Strength (psi)								
1 Day	511	383	466	399	519	468	386	300
28 Day	678	582	639	620	659	647	561	459
Weights (lb/ft^3)								
7 Day	129.9	129.3	128.5	128.8	125.2	126.9	124.3	117.9
28 Day	125.6	123.0	123.8	123.8	121.4	122.3	117.6	111.6
Equilibrium	122.0	120.0	120.3	119.6	118.1	118.3	115.6	109.6

Table C.1 (cont.) More Results for Mix Designs in Initial Iteration

	Repeats		8000 psi Mixes				Hybrid
Mix Number	**5-2**	**6-2**	**H1**	**H2**	**H3**	**H4**	**9**
Compressive Strength (psi)							
1 Day	4220	4218	4710	3785	4961	3201	4469
3 Day	5635	5360	5697	5795	5842	5018	5871
7 Day	5860	5624	6044	6339	6521	5589	6346
28 Day	6910	6637	7130	6905	6973	6126	6851
Modulus of Elasticity (ksi)							
1 Day	2074	2014	2156	2023	2084	1840	1984
28 Day	2857	2473	2670	2784	2539	2297	2600
Modulus of Rupture (psi)							
1 Day	n/a	n/a	448	445	485	464	n/a
28 Day	n/a	n/a	713	703	693	643	n/a
Splitting Strength (psi)							
1 Day	436	369	343	263	329	244	439
28 Day	606	567	553	594	574	581	562
Weights (lb/ft^3)							
7 Day	123.8	122.3	124.2	125.4	123.3	123.4	123.6
28 Day	118.9	116.9	119.7	120.3	119.1	n/a	118.8
Equilibrium	117.6	117.7	117.4	116.4	118.1	116.0	117.8

Table C.2 Results for Mix Designs in Second Iteration

Mix Number	1-C	1-W	1-S	2-C	2-W	2-S
Compressive Strength (psi)						
1 Day	4395	4549	4643	4985	4463	4527
3 Day	5532	5574	5845	6133	5420	6182
7 Day	6008	5730	6432	6579	5609	N/A
28 Day	7125	6441	7812	7685	6185	8023
Modulus of Elasticity (ksi)						
1 Day	2217	1980	2297	2253	2068	2742
28 Day	2755	2828	2297	2845	2757	2742
Modulus of Rupture (psi)						
1 Day	475	490	523	483	460	508
28 Day	658	668	735	625	655	650
Splitting Strength (psi)						
1 Day	437	394	397	440	382	398
28 Day	606	507	531	595	482	520
Weights (lb/ft^3)						
Fresh	126.5	127.9	121.4	126.1	126.8	119.5
7 Day	126.5	127.3	121.4	127.0	126.6	
28 Day	121.7	122.0	117.4	127.2	126.5	
Equilbrium	119.3	119.1	115.1	121.1	119.8	114.2

Table C.2 (cont.) More Results for Mix Designs in Second Iteration

Mix Number	3-C	3-W	3-S	4-C	4-W	4-S	5-C	5-S
Compressive Strength (psi)								
1 Day	5164	5141	4599	5448	5054	4980	4544	3600
3 Day	6234	6015	5908	6479	5633	6119	6217	5151
7 Day	6874	6312	7305	6799	5857	6681	6001	6051
28 Day	7839	6677	8103	8037	6660	7935	7441	7216
Modulus of Elasticity (ksi)								
1 Day	2212	2383	2273	2250	2354	2242	2075	1125
28 Day	2791	3186	2814	2797	2989	2773	2637	2199
Modulus of Rupture (psi)								
1 Day	430	465	475	525	484	448	420	420
28 Day	658	675	673	753	599	665	655	653
Splitting Strength (psi)								
1 Day	392	461	431	491	429	392	380	328
28 Day	689	544	587	590	483	499	555	472
Weights (lb/ft^3)								
Fresh	126.4	125.6	122.3	127.8	117.9		123.1	117.6
7 Day	126.1	124.8	122.8	126.9	120.8	117.1	123.5	117.8
28 Day	122.2	121.4	118.7	122.7			117.5	112.9
Equilbrium	119.7	118.9	117.2	120.6	115.8	111.3	116.1	111.7

Table C.3 Results for Mix Designs in Third Iteration

Mix Number	F-1	F1 4x8	F-2	F2 4x8	F-3	F3 4x8	F-4	F4 4x8
Compressive Strength (psi)								
1 Day	3060	2952	2946	2717	3061	2733	3554	3265
3 Day	4202	4503	4031	3974	4188	4252	4647	4401
7 Day	5029	N/A	4728	4706	4827	4947	5423	5130
28 Day	6443	6571	6272	6156	6492	6177	6760	6629
Modulus of Elasticity (ksi)								
1 Day	1980	N/A	1996	N/A	2140	1903	2099	1891
28 Day	3017	2853	3095	3109	2907	2840	2904	3029
Splitting Strength (psi)								
1 Day	290	259	222	272	236	253	277	291
28 Day	587	N/A	518	561	527	548	498	518
Weights (lb/ft^3)								
Fresh	122.4		122.7		122.4		121.6	
7 Day	123.3		123.8		123.7		122.4	
28 Day	117.7		116.6		116.3		116.4	

Mix Number	F-5	F5 4x8	F-6	F6 4x8	F-7	F7 4x8	F-8	F8 4x8	F-9	F9 4x8
Compressive Strength (psi)										
1 Day	4393	3899	5525	5019	6092	5609	5610	5236	5856	5216
3 Day	5297	5038	6454	6139	7225	6853	6885	6703	6871	6534
7 Day	6261	6038	6953	6663	7715	7717	7379	7724	7502	7569
28 Day	7449	7184	7892	7973	8420	8632	8391	8626	8100	8432
Modulus of Elasticity (ksi)										
1 Day	2315	N/A	2321	2226	2273	2306	2520	N/A	2498	2514
28 Day	3053	2169	3147	3324	3385	3611	3390	3722	3396	3285
Splitting Strength (psi)										
1 Day	313	328	313	373	318	377	326	409	313	394
28 Day	614	593	564	568	452	575	491	581	491	604
Weights (lb/ft^3)										
Fresh	122.4		123.5		124.9		124.5		124.3	
7 Day	123.5		123.4		125.4		125.2		124.6	
28 Day	117.9		119.5		122.3		121.2		120.6	

Appendix D: Concrete Strain Profiles

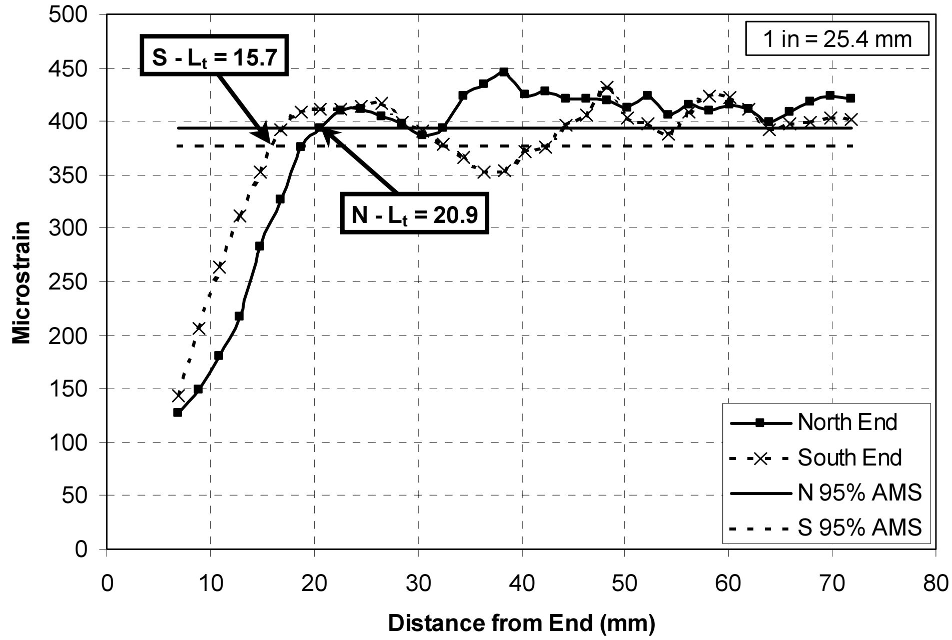

Figure D.1 Strain Profiles for NW6000-1

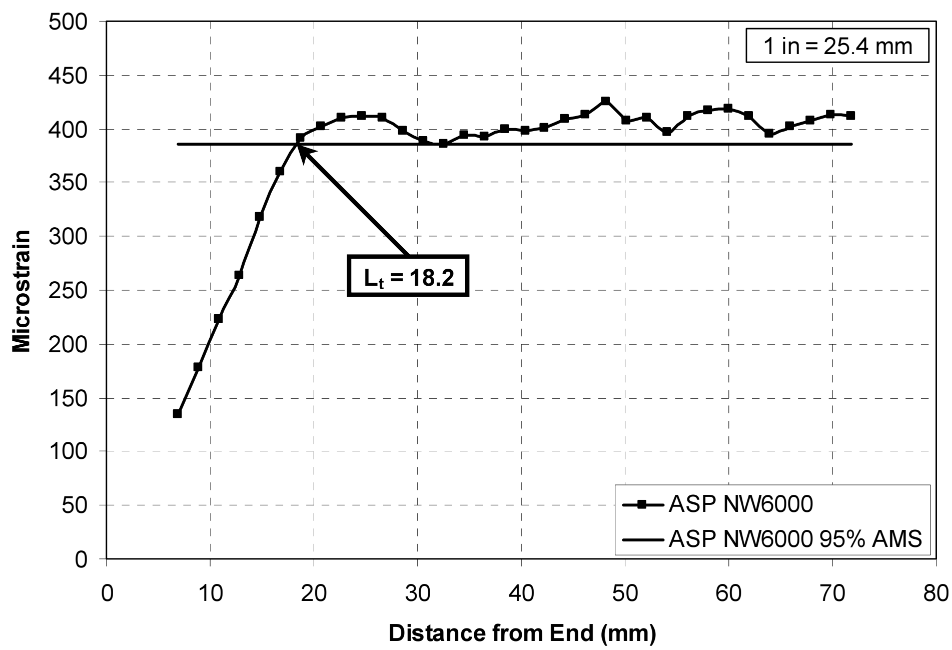

Figure D.2 Average Strain Profile for NW6000

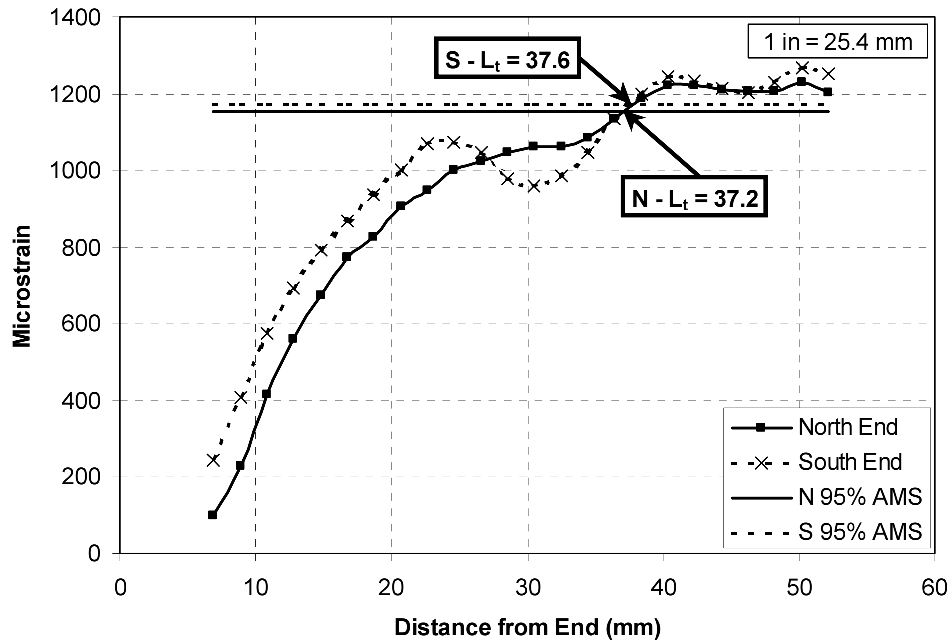

Figure D.3 Strain Profiles for LW6000-20

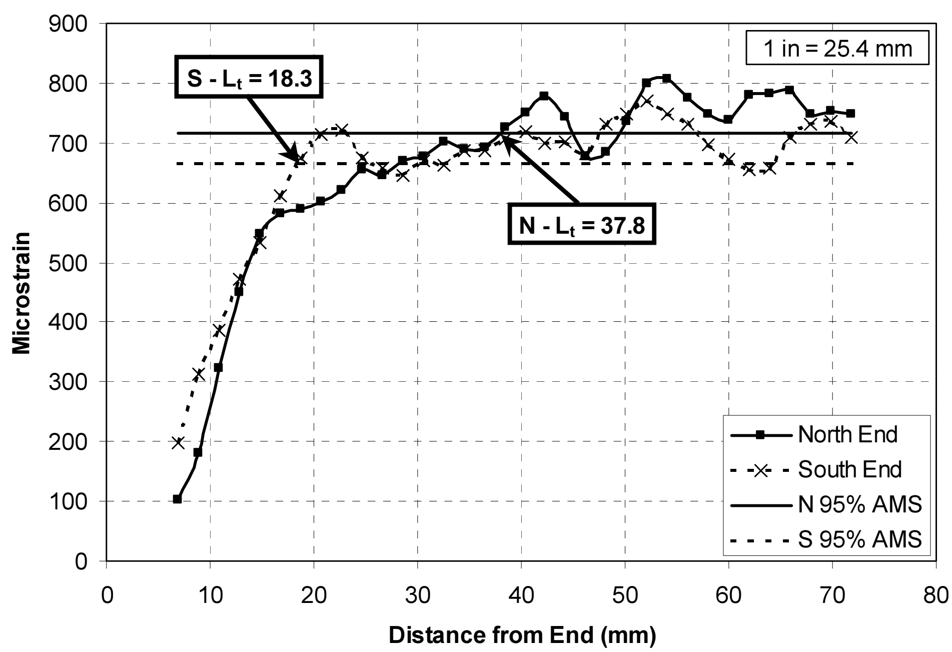

Figure D.4 Strain Profiles for LW6000-1

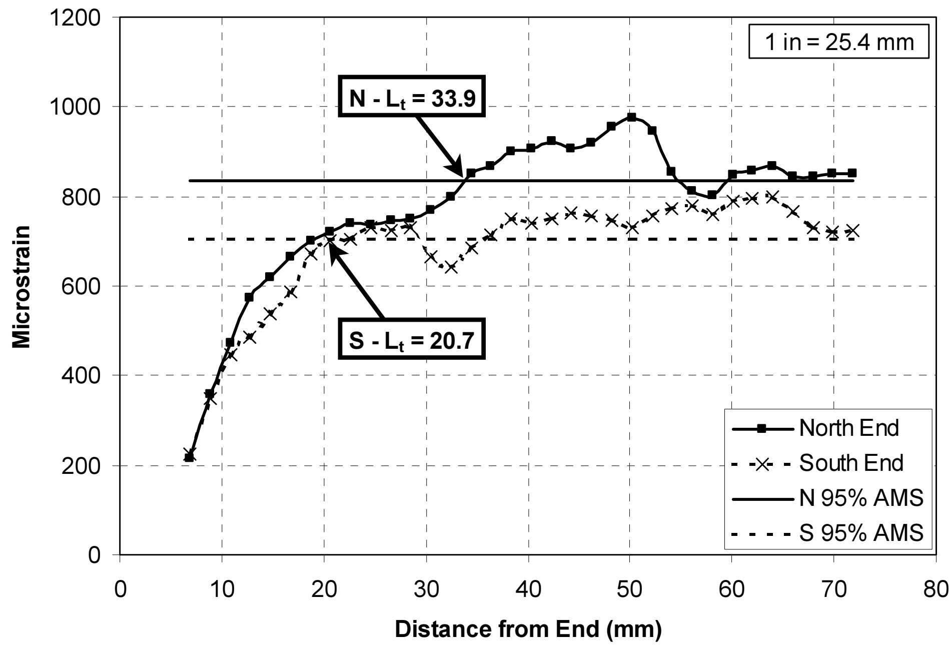

Figure D.5 Strain Profiles for LW6000-2

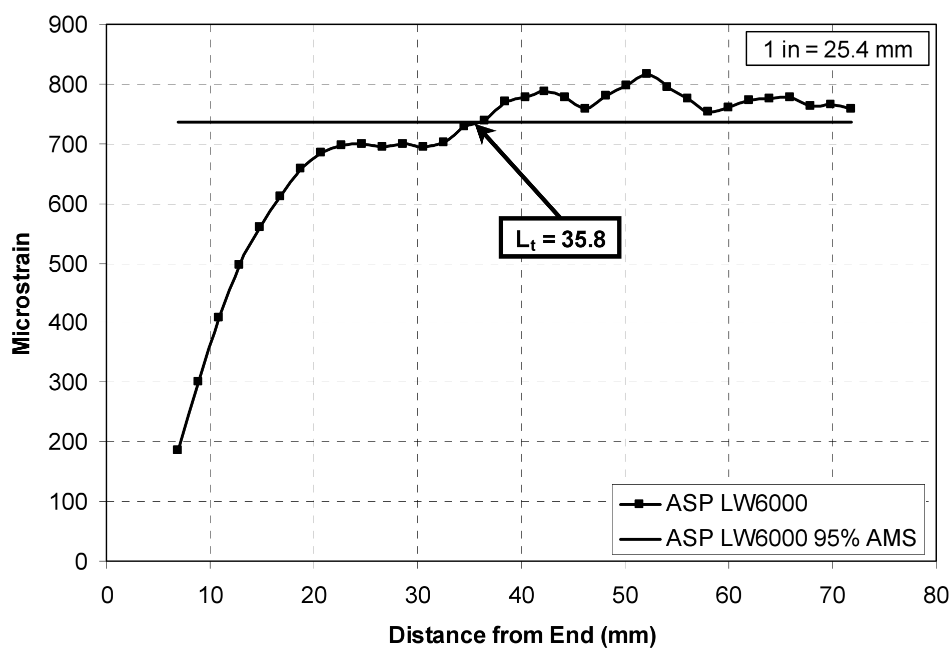

Figure D.6 Average Strain Profile for LW6000

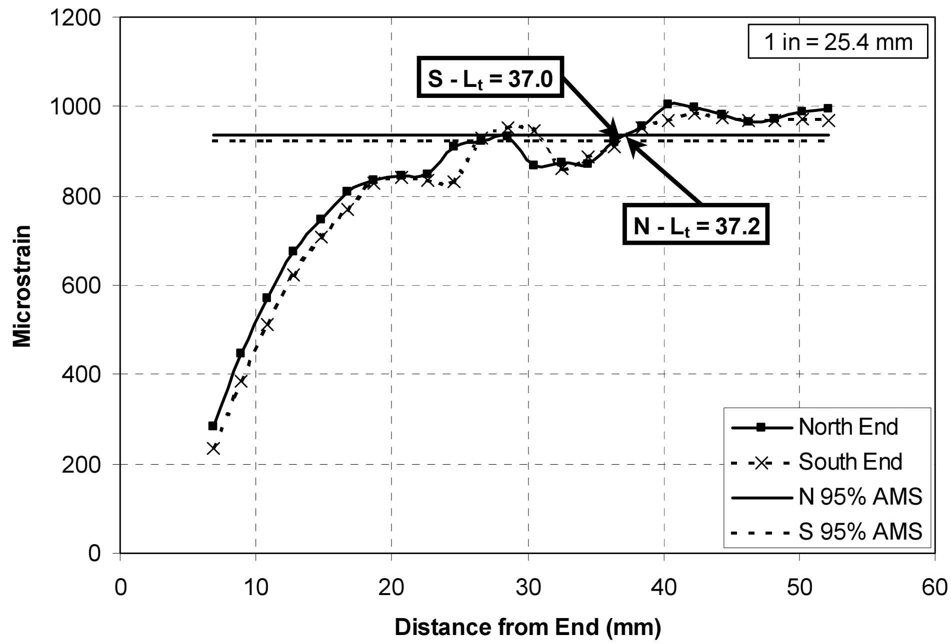

Figure D.7 Strain Profiles for LW8000-20

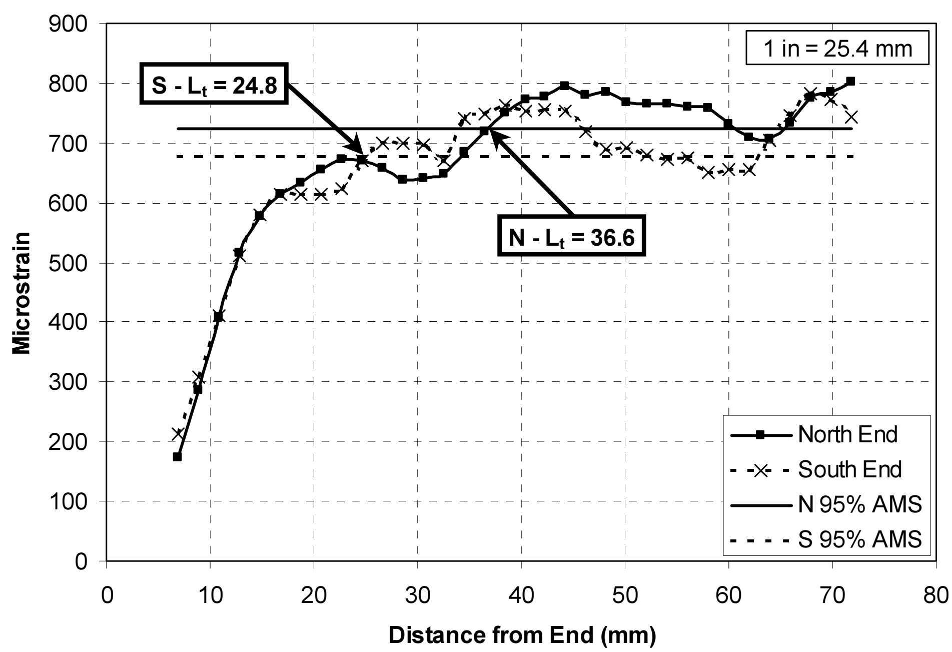

Figure D.8 Strain Profiles for LW8000-1

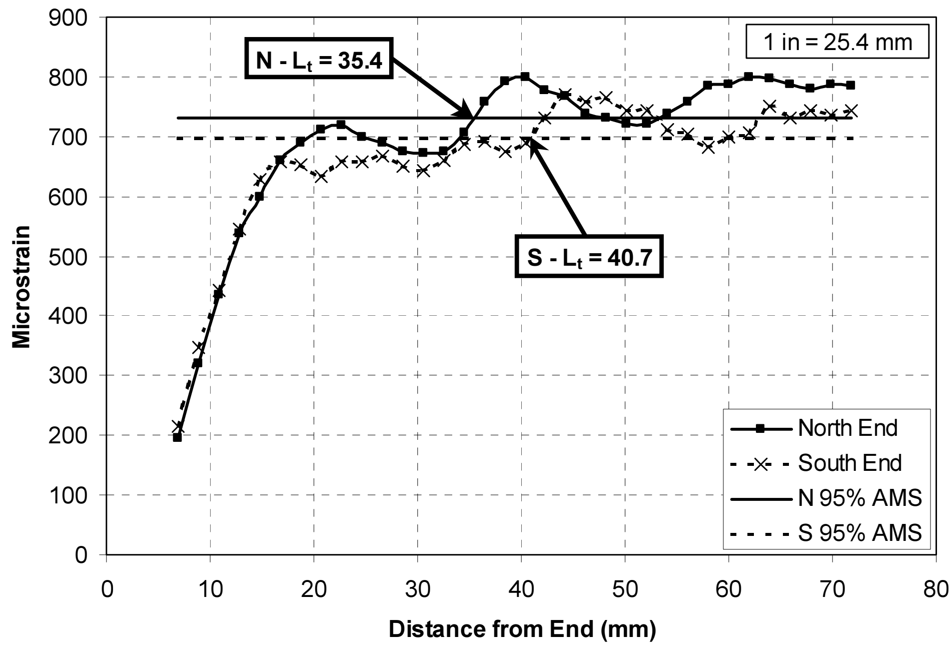

Figure D.9 Strain Profiles for LW8000-2

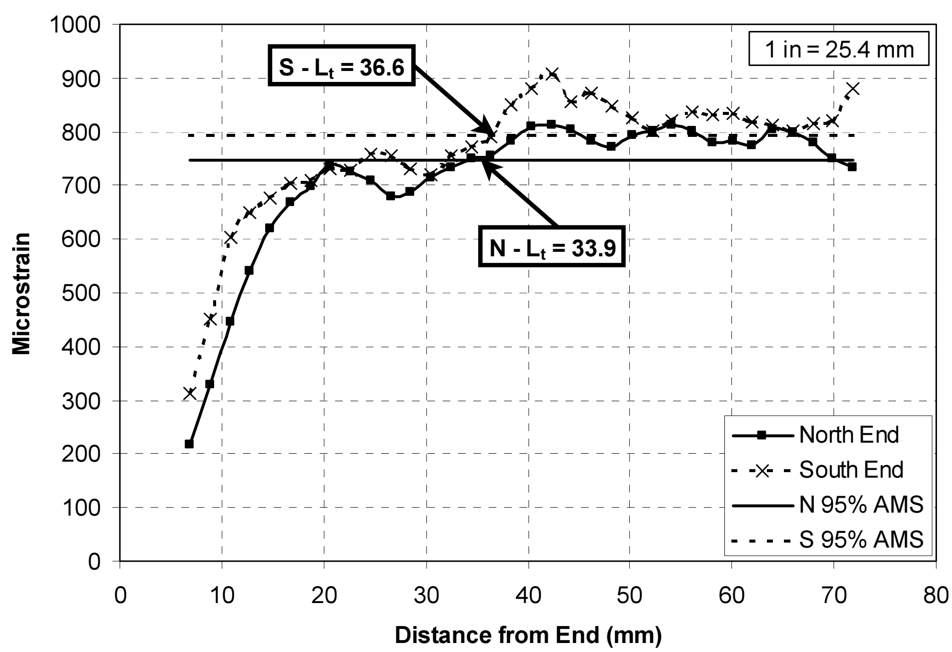

Figure D.10 Strain Profiles for LW8000-3

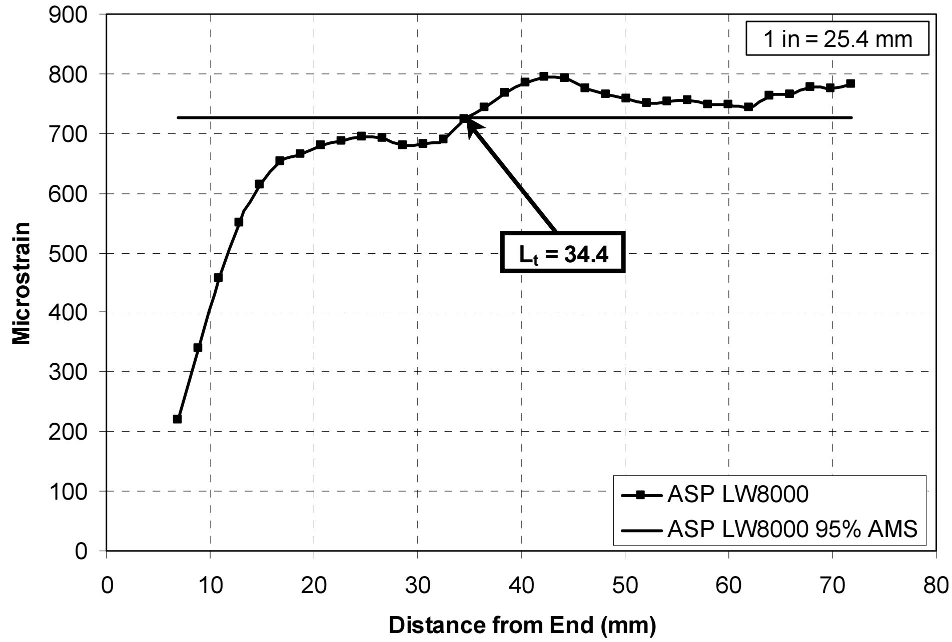

Figure D.11 Average Strain Profile for LW8000

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