Feasibility of Utilizing High Performance Lightweight Concrete in Pretensioned Bridge Girders and Panels

by

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Ned H. Burns, Supervisor

John E. Breen, Supervisor

Dedication

To my wife, Alice Marie, and our twin children, Stephanie and Stephen

for all their love, encouragement, and support.

We did it again!

Acknowledgements

Thanks to God for giving me the strength and courage to pursue and complete this research. I also thank God for giving me the opportunity to work with two of the very finest, Dr. Ned H. Burns and Dr. John E. Breen. I will always have the deepest gratitude and respect for both these gentlemen. We are truly blessed to have these men in our profession and in our lives. Thanks for all your guidance.

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Without the direct and indirect support of many other people, projects like these would be very difficult. Thanks to everyone at the lab for their assistance and to all the administrative staff that work for the Ferguson Structural Engineering Laboratory and the UT Civil Engineering Department for all their hard work.

Sincere appreciation to all the construction industry companies and personnel that made completion of this research possible by providing valuable information as well as assistance.

December 7, 2001

Abstract

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The use of high performance lightweight concrete in Texas prestressed concrete bridges has potential advantages and disadvantages. Advantages include reduced dead load, crane capacity, and shipping costs. Disadvantages include higher prestress losses, deflections, camber, and material costs.

Prestressed concrete bridge girders can be designed with lightweight concrete that has compressive strengths of 6000 psi and 7500 psi and unit weights of 118 pcf to 122 pcf, respectively. Comparisons of AASHTO Type IV girders made from normal weight concrete and girders made from lightweight concrete, both with various composite concrete deck combinations, reveal that higher prestress losses and lower allowable stresses reduce the possibility of having fewer prestressing strands in the lightweight girder. The design of the lightweight

concrete girder was controlled by the allowable stresses and not by strength requirements. The lower modulus of elasticity of lightweight concrete results in higher camber and deflections.

Testing of 3/8-inch prestressing strands in precast concrete panels for transfer length showed that the AASHTO provision of 50 times the strand diameter is conservative. The transfer length in the lightweight concrete panel was slightly higher than the transfer length in the normal weight concrete panels, but both were below the AASHTO requirement.

Lightweight concrete material costs are higher than normal weight concrete. However, the higher costs are somewhat offset by reduced shipping costs. Larger shipping savings for girders can be realized by shipping two girders at the same time, but this is only practical for the smaller Type A girders. The precast concrete panels made from lightweight concrete also provide opportunity for reducing the shipping and handling costs.

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CHAPTER 1
Introduction

1.1 BACKGROUND

The development of concrete or at least one of its dominant constituents, cement, dates back to a time before the Romans. The Romans who built such notable structures as the Coliseum and Pantheon in Rome, as well as the Pont du Gard aqueduct in south France used pozzalana cement in the construction of these structures between the years of 300 BC and 476 AD. Although the Romans have made very notable structural contributions, including several bridges that are still standing today, there is evidence that cements were discovered and in use even before their time. In the years following and up through the 1800's, several individuals from different parts of the world continued developing and patenting cement or mortars, but it was not until 1824 that Joseph Ardin invented Portland cement, named after stones quarried in Portland, England. This was followed in 1889 by the construction of the first reinforced concrete bridge and in 1891 by the placement of the first concrete street in Bellefontaine, Ohio. In 1903, the first concrete high rise building was constructed in Cincinnati, and the use of concrete as a major building material was continued with the construction of Hoover Dam and Grand Coulee Dam. Even today, concrete continues to be a predominant building material that defines and gives shape to our structural endeavors, be it a bridge, dam, building, or a roadway. The National Bridge Inventory (NBI) reveals that as of August 2000, more than 75 percent of the bridges in existence in Texas are of concrete, including both reinforced and prestressed concrete.

Concrete is a very unique material because it can be tailored to meet the specific needs of any project, within the limits of structural adequacy. The tailoring can vary from changing the shape of the members for the purpose of providing the most economical or most visually pleasing design, to varying the composition of the concrete mixture to meet structural, environmental, construction, and/or economic conditions.

Concrete is a mixture of mainly four basic materials. These materials include a coarse aggregate, typically gravel or crushed stone; a fine aggregate, typically sand; cement; and water. Along with the four basic materials, concrete may also include admixtures such as retarders or superplastizicers that alter the properties of the mix to achieve desired project results.

Project 0-1852, Prestressed Structural Lightweight Concrete Beams, sponsored by The Texas Department of Transportation, was commissioned to examine the potential use of structural lightweight concrete in typical precast concrete I-girder bridges. The lightweight concrete is achieved by altering of the mix design to use a much lighter pyroprocessed material, such as an expanded clay or shale, to replace the heavy coarse aggregate. The direct impact of using this lighter material is that the overall dead load of a structural member is reduced to approximately 80 percent of the weight of a concrete member made from concrete that utilizes the heavier coarse aggregates such as gravel or crushed stone. The use of lightweight concrete in the United States is not a new concept and its use can most likely be attributed to the shipbuilding industries' use of this material in 1918 [1]. However use of this material is not just limited to the ship building industry. There have been several successful bridge projects constructed around the world, including the United States, which have utilized lightweight concrete. Even though this material has seen limited use since its early beginnings, it is possible that with knowledge gained from additional research on this material that lightweight concrete in the future could be a competitive material for prestressed concrete bridge construction.

1.1.1 Prestressed Concrete

Because of concrete's inherent characteristic of being weak in tension, the development of prestressed concrete, attributed to E. Freyssinet of France, is certainly a noteworthy discovery. In the prestressing of concrete, compressive stresses are induced into a concrete member to counteract the tensile stresses produced by the member's self-weight and those due to superimposed loads. Freyssinet was successful in producing a prestressed concrete bridge through the use of high strength steel wires. Earlier attempts by others at prestressing with lower strength steels were unsuccessful because the prestressing induced on a member was lost due to shrinkage and creep of the concrete. The high-strength steels, such as those in use today, can be subjected to larger strains, larger than the strains produced by shrinkage and creep of the concrete.

The beginning of prestressed concrete in the United States is marked by the construction of the Philadelphia Walnut Lane Bridge in 1949. Ever since then, the use of prestressed concrete bridges has increased and has almost become an exclusive standard for bridges in Texas with spans less than about 125 to 135 feet. Another important aspect of prestressed concrete is that because it is usually plant-cast and usually has low water/cement ratios the concrete will be more durable than site cast concrete [2]. Durability of concrete is an important aspect in reducing maintenance costs and increasing life expectancy of any structure. As mentioned before, approximately 75 percent of all bridges in Texas are made from either reinforced or prestressed concrete according to National Bridge Inventory information. Prestressed concrete represents about 20 percent of all bridges in Texas. Another important aspect to consider regarding bridges in Texas, is that according to the NBI approximately 7 percent of all bridges are structurally deficient and approximately 15 percent are functionally obsolete. In considering possible replacements or rehabilitation of these structures, it is possible that pretensioned members made from structural lightweight concrete might be a viable alternative to normal weight concrete for the reconstruction needed.

1.1.2 Lightweight Concrete

According to the Expanded Shale, Clay, and Slate Institute: "For nearly a century ESCS (Expanded Shale, Clay, and Slate) has been used successfully around the world in more than 50 different types of applications. The most notable among these are concrete masonry, high-rise buildings, concrete bridge decks, precast and prestressed concrete elements, asphalt road surfaces, soil conditioner, and geotechnical fills." [3]. As previously mentioned, one of the first uses of lightweight concrete in the United States was in the construction of World War I ships by the Emergency Fleet Building Corporation. Another early use of lightweight concrete was construction of the upper deck of the San Francisco-Oakland Bay Bridge in 1930. As of 1980, the lightweight concrete deck on this bridge was reported to still be in service with only minimal maintenance. It is further reported that the lightweight deck was one of the keys to the economic feasibility of this bridge. More recently, the majority of bridge construction utilizing lightweight concrete has been overseas, in countries such as Norway. In the United States, some projects other than the San Francisco-Oakland Bay Bridge that have utilized lightweight concrete include the Whitehurst Freeway in Washington D.C., the Suwanee River Bridge at Fanning

Springs and the Sebastian Inlet Bridge. The last two bridges were both built by The Florida Department of Transportation.

1.2 OBJECTIVES

The main objective of this project, Project 0-1852, is to determine the feasibility of using high performance lightweight concrete in bridge girders and deck panels. Originally, only bridge girders were included in this study, but the scope of the research was expanded to also evaluate the viability of using precast concrete panels made from lightweight concrete as well. This project was subdivided into several tasks that are as follows:

Task 1) Literature Search
Task 2) Past Use of Lightweight Concrete in Texas
Task 3) Develop Concrete Mix Designs
Task 4) Materials Research & Testing
Task 5) Full Scale Testing of Type A Beams with Decks
Task 6) Prestress Loss and Evaluation of Beam Behavior/Handling of Beams/Final Report

Most of these tasks have been completed and are documented in theses by Heffington, Kolozs, and Thatcher [References 4,5, and 6, respectively].

1.3 SCOPE

The focus of this report will be to utilize properties of the lightweight concrete tested in this project to evaluate the feasibility of utilizing it for the fabrication of pretensioned precast bridge girders and panels. Feasibility of the lightweight concrete will be accomplished by performing several analyses using The Texas Department of Transportation's program for designing prestressed concrete girders. This program, commonly known as PSTRS14, will be used to analyze both normal and lightweight concrete girders and then a comparison of results from this analysis will be performed. Also as part of the feasibility determination, a cost comparison will be performed between using normal and lightweight concrete. The cost data will be obtained from industry sources familiar with these materials. Finally, also included in this report will be a discussion on the transfer length of 3/8-inch prestressing strand used in the precast concrete deck panels. This testing was performed on the 3/8-inch strand to insure that the transfer length in a panel made from lightweight concrete would be sufficient.

1.4 ORGANIZATION

This thesis is divided into 5 chapters. Chapter 1 provides background information for concrete including lightweight concrete. A discussion of the findings regarding the transfer length of 3/8-inch strand in precast concrete panels is found in Chapter 2, while the beam analysis utilizing TxDOT's PSTRS14 Program is presented in Chapter 3. Chapter 4 will concentrate on presenting information regarding material availability as well as economic cost information for lightweight concrete. Also discussed in this chapter will be design guidelines. Finally, Chapter 5 will be a summary of the findings as well as recommendations for implementation, which will conclude the report.

CHAPTER 2

Panel Transfer Length Testing

2.1 INTRODUCTION

According to the TxDOT Bridge Design Guide, "Precast prestressed concrete panels are the preferred method of constructing decks on prestressed concrete beams and are used occasionally on steel beams and girders."[7] This method of construction, shown in Figure 2.1, was developed in Texas during the early 1960's and has been widely used throughout the state because it eliminates a considerable portion of the formwork required for constructing the composite slab. Another advantage is that it provides an instantaneous surface that can be used immediately in the construction of the cast-in-place deck.

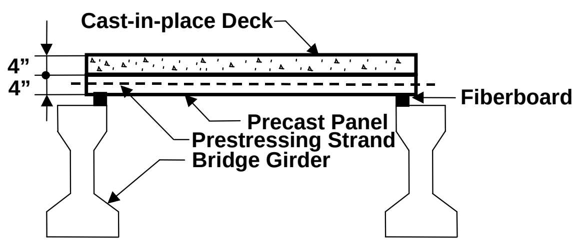

Figure 2.1 Precast Panel Stay-in-Place Forms

In the past, panels have traditionally been cast from normal weight concrete, but the scope of this project was amended to include an investigation of use of lightweight concrete as an alternative material for constructing the panels. The lightweight beam tests completed by Kolozs [5], indicated that transfer lengths for the pretensioning strands in the beams were longer than expected.

This raised the question of whether or not the fairly short 3/8-inch pretensioning strands in a lightweight panel would have sufficient transfer length. The purpose of this report is to present information and conclusions regarding the transfer length testing of six precast concrete panels.

2.2 TEST SETUP

Three normal weight and three lightweight precast concrete panels were cast. The normal weight panels are identified as D52, D53, and D54, while the lightweight panels are D55, D56, and D57. All panels were cast at the same time by a supplier of precast products very familiar with these types of panels. In fact, these panels were cast on the same line as others being fabricated for an upcoming bridge project. Hence, they were placed, finished, and cured exactly the same as other panels being fabricated for an actual project. The only difference was that the lightweight concrete was obtained from a offsite local ready-mix supplier, while the normal weight concrete was a plant mix batched on site.

2.2.1 General Layout of Panel

The physical dimensions of a typical panel are shown in Figure 2.2. Also shown in this figure is the general location where the DEMEC (demountable mechanical) strain gauge reference points used for the measurements were placed. The placement of the reference points was parallel to the direction of the pretensioning strands at offsets of 4 feet and 2 feet from the edge of the panel. These correspond to the centerline and ¼ point, respectively. Two basic arrangements of reference points were used in the testing and the arrangement for each panel is as noted in Table 2.1.

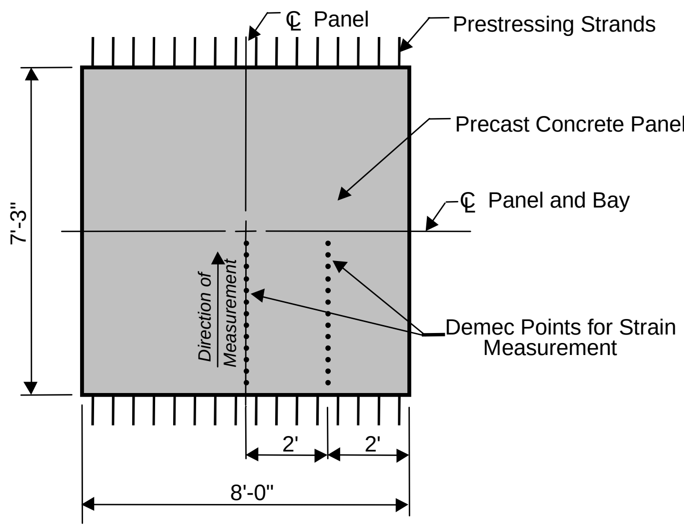

Figure 2.2 Precast Panel Layout

Table 2.1 Panel Transfer Length Specimens

Panel ID	Concrete Type	DEMEC Points at CL	DEMEC Points at ¼ Pt
D52	Normal Weight	✔	
D53	Normal Weight	✔	✔
D54	Normal Weight	✔	✔
D55	Lightweight	✔	
D56	Lightweight	✔	✔
D57	Lightweight	✔	✔

2.2.2 Instrumenting of Panel

2.2.2.1 DEMEC Strain Gauge

All strain measurements were performed with the DEMEC strain gauge shown in Figure 2.3. This extensometer is outfitted with a Mitutoyo digital gauge and has a 200-mm gauge length. The same gauge was used consistently throughout the measurements to eliminate possible differences amongst gauges. Also shown in the figure, is the set out bar (darker colored bar with points) and the Invar bar used to zero the gauge. The set out bar was used to apply the strain reference points so as to be as close to the gauge length of the DEMEC extensometer as possible. This would insure that once the pretensioning strands were released and the panel would become compressed that the movement of the points would still be within the allowable measuring range of the DEMEC.

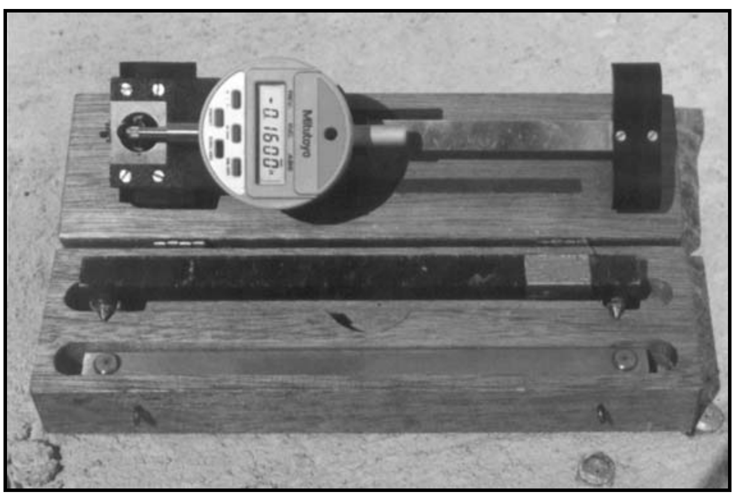

Figure 2.3 Digital DEMEC Strain Gauge

2.2.2.2 Reference Point Fabrication

The points used for strain measurements were fabricated in the Ferguson Structural Engineering Laboratory and were similar to ones used in other projects. The points were prepared by drilling a small hole on the head of a ¼-inch dia. x 1-inch long Hilti Metal HIT anchor as shown in Figure 2.4. This hole, which would accept the locating points of the DEMEC gauge, would serve as the reference guide for measurements. For the purpose of allowing possible adjustments in the field to account for misalignment, the hole drilled on the head of the anchor was offset from the center. This would allow rotation of the anchor during placement so that the distance between the reference points would be within the limits of movement of the DEMEC extensometer locating points.

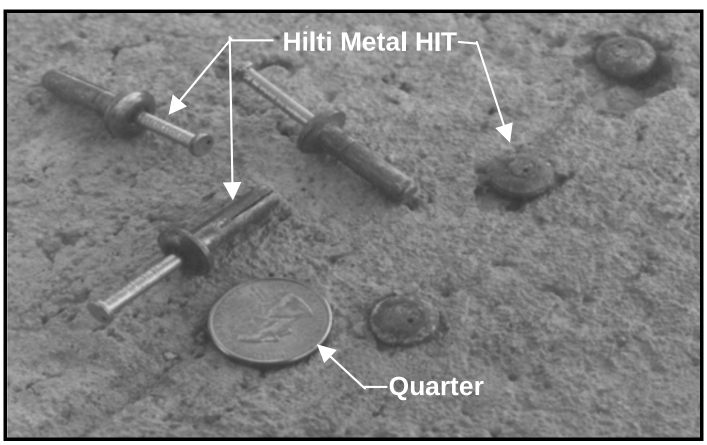

Figure 2.4 Anchors Modified for Use as Strain Reference Points

2.2.2.3 Reference Point Installation

After the panels were placed and allowed to cure for approximately 18 hours, the fabricated reference points were installed in the panel. The installation began by drilling holes into the precast panels every 1.97-inch (50mm) using Hilti Rotary Hammer drills with ¼-inch drill bits. Because of panel symmetry, reference points were installed in only half of the panel. The spacing of the drilled holes on the top of the panel was maintained with steel templates made from rectangular hollow tubing that was predrilled in the laboratory to the required hole spacing. The template served as a guide in maintaining both the horizontal and vertical control of the holes.

Drilling into the lightweight concrete was easier than drilling into the normal weight concrete. It was also observed that the panels cast from the lightweight concrete were still somewhat “moist” after nearly one day of curing. This was evident from the cuttings that became “pasty” or “mud-like” during the drilling operation. The normal weight concrete cuttings were considerably more “powdery” and “dusty”.

Once the drilling was completed, placement of the reference points began. As an added measure to prevent any possible movement of the strain reference point, it was planned to use an epoxy adhesive to supplement the wedging action of the anchor. However, the use of this epoxy adhesive proved to be a problem because the type chosen did not allow enough time for positioning of the points. Positioning of the points was an intricate and time-consuming procedure because each point had to have the offset hole in the head of the anchor rotated into a position that would be within the limits of the DEMEC strain gauge. This was done by using the setting out bar included with the DEMEC gauge. After the correct distance was established the anchor was partially tapped into the drilled hole and the distance was rechecked. This procedure was continually repeated for each point until they were completely seated on the top of the panel. Because this procedure took so long, it was decided to forgo the use of the epoxy adhesive. Figure 2.5 represents a cross-sectional view of a manufactured DEMEC reference point in place on the top of a precast concrete panel.

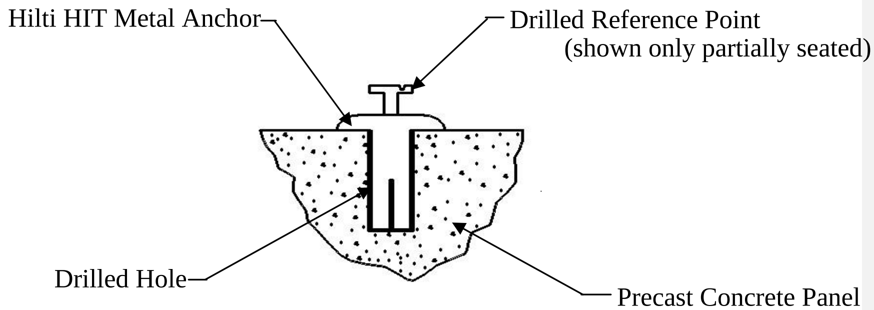

Figure 2.5 Placement of Strain Reference Point

2.2.2.4 Materials

Two types of concrete were used in the precasting of the panels, a normal weight and a lightweight. The normal weight concrete was batched by the precast manufacturer on-site, while the lightweight was obtained from a local ready-mix supplier who also delivered it. In Table 2.2, the results from the compression tests performed on 6-inch x 12-inch cylinders prepared for each of the concrete types is provided.

Table 2.2 Concrete Compressive Strengths

Material Type	Cylinder No.	Time of Testing	Measured Compressive Strength	Average Compressive Strength
		(days)	(psi)	(psi)
NW	1	7	8050	8575
NW	2	7	9100	
LW	1	7	5075	5112
LW	2	7	5150	
NW	1	32	9550	9600
NW	2	32	9650	
LW	1	32	5625	6212
LW	1	32	6800	

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From the results shown, it is evident that the normal weight mix was a very high strength mix, while the strength of the lightweight concrete was rated by the supplier as 5000 psi at 28 days (mix design for the lightweight concrete can be found in the appendix). The only requirement that was placed upon the supplier was that the lightweight mix be a little drier than that sent out to a previous research project where lightweight panels were also cast. That mix was very wet and achieving the required strength at release of these panels was a concern. The supplier adjusted the mix design by reducing the amount of superplastizer from 15 ozs/100cwt to 8 ozs/100cwt and by slightly lowering the retarder to maintain 2.5 ozs/100cwt. Due to these changes, the mix was placed without any difficulties and there appeared to be no difference in placement between the normal and lightweight concrete.

2.3 TEST PROCEDURE

Prior to release of the pretensioning strands for the panels, strain measurements were taken for all six panels. Before beginning measurements on each line of strain reference points, the digital DEMEC device was zeroed on the Invar bar. Readings were taken at every point, which were spaced almost 2 inches apart, beginning at the top edge of each panel and working toward the center of the panel. Each reading for a single point was repeated until a duplicate reading was obtained. For instance, if the first reading on the DEMEC device was .0120 inches and the second reading was .0120, then no additional readings were taken. However if the second reading was .0115, then a third reading was taken. Usually three readings were enough to establish an identical measurement. The identical readings were accepted as the correct measurement.

After completing the readings for all points on the panels, the pretensioning strands for the entire precasting line was released. Because the research panels were on the opposite end from where separation of each panel was taking place, a flame-cutting device was used to cut the pretensioning strands to separate each of the these panels. Upon complete release of each individual panel, measurements for each point were then again repeated using the same procedure described above. Readings were again repeated for all the panels approximately 85 days later. After the readings at 85 days were completed, it was believed that sufficient data had been obtained to determine the transfer length of the 3/8-inch pretensioning strand in these typical sized panels, hence the next step was to reduce and analyze the data.

2.4 TEST RESULTS

2.4.1 Data Reduction

After all readings that included readings before release, after release, and 85 days later were completed, the data was reduced by taking each measurement after release and subtracting it from the corresponding measurement before release. This difference was the change in length experienced by the panel at that location due to release of the pretensioning strands. However, to obtain the strain, this change in length was then divided by the gauge length (200-mm) of the DEMEC strain gauge. These same data reductions were done for readings taken at 85 days.

2.4.2 Data Smoothing

Because of scatter in data due to reading imperfections as well as possible material moduli differences within a panel, the plots of strain versus distance produced profiles with considerable variability. In order to obtain a smoother profile of strain versus distance, two different smoothing techniques as utilized previously by Kolozs on this project were also utilized for this data [5]. The first technique involves the averaging of three consecutive strain measurements and then applying that single average, $\varepsilon_{i,smooth}$, at the center of the points. This method is graphically displayed in Figure 2.6.

The other method for reducing variability simply involved taking the "smoothed" strain measurements for the centerline and again averaging them with the "smoothed" strain measurements from the edge. This would reduce the variability of strains at the center and edge of the panel. Also, because panels

D53, D54, D56, and D57 were the only panels with reference points at both the centerline and ¼ point, only the data for these panels were “averaged”.

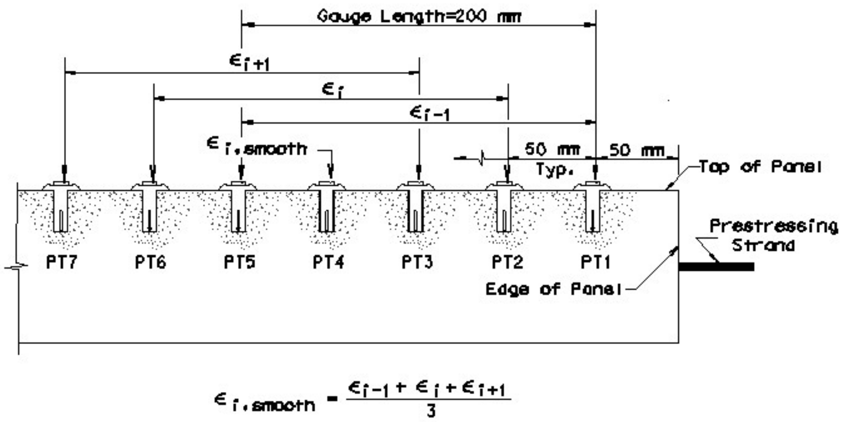


Figure 2.6 Smoothing of Strain Points

2.4.3 Data Results

From the values determined after application of the smoothing and averaging methods described in the previous section, two separate figures were prepared. These figures represent strain versus distance along the panel. Figure 2.7 represents the smoothed and averaged data for measurements taken immediately before and after release, and Figure 2.8 is for the data measured approximately 85 days later.

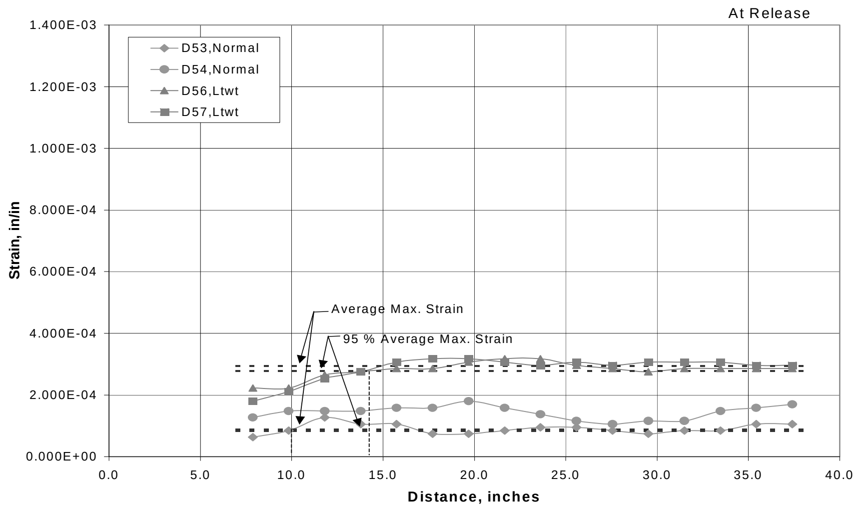


Figure 2.7 Panel Transfer Length Strain Measurements at Release

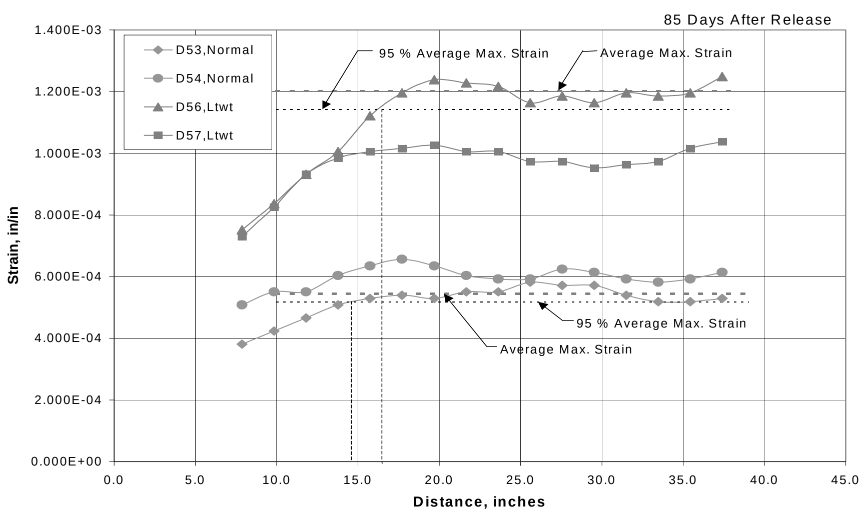


Figure 2.8 Panel Transfer Length Measurements at 85 days

The characteristic behavior expected from these plots of strain versus distance is that the data points will steadily increase, representing increasing levels of stress along the length of the strand, and then the data will plateau at the point where the stress becomes constant. The transfer length will then be determined by taking the distance from where the stress is zero, hence the edge of the panel, to the point where the stress becomes constant. Because the point of constant stress is sometimes not very well defined, a method used in previous experiments [5] will also be used here. This method reduces some of the subjectivity and is commonly known as the "95% Average Maximum Strain" method as shown in Figures 2.7 and 2.8. This method is applied by averaging all points on the plateau. The average of all these points is termed the "average maximum strain". Next, a horizontal line is plotted through the point that is 95 percent of this "average maximum strain". Once this is obtained, the intersection of a horizontal with the ascending portion of the strain versus distance data points represents distance required to fully transfer the prestressing upon release of the strands.

2.5 DISCUSSION OF TEST RESULTS

Both Figure 2.7 and Figure 2.8 consistently indicate that the strains for the lightweight concrete are approximately twice as large as the strains for the normal weight concrete. This is mainly due to the lower modulus of elasticity typical of lightweight concrete. It is also evident from these figures that the strains in each panel have increased approximately fourfold in a time period of about 85 days, with both materials displaying similar increases in strains. However, because the overall difference in strains (2 E-04 in/in) for the normal weight concrete is less than half the overall difference of the strains (4.5 E-04 in/in) for the lightweight concrete, it can be rationalized that the stresses for the normal weight concrete are more uniform along the length of the strand.

Despite the differences noted, the transfer length determined by the 95 percent average maximum strain for each of the concrete types did not differ by more than about 10 percent, with the lightweight concrete requiring the largest transfer length. This required length is equivalent to about 45 strand diameters (d_S), while the required length for the normal weight concrete was approximately 39 d_S. Both of these transfer lengths are less than the 18.75 inches that would be given using the AASHTO Section 9.20.2.4 criteria of 50 times the strand diameter.

In conclusion, 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-inch strands measured in this test for both normal weight and lightweight concrete is less than that predicted using AASHTO transfer length criteria. Further, the transfer length in panels made from lightweight concrete is only slightly (10 percent) more than in panels made from normal weight concrete. The same AASHTO Section 9.20.2.4 design rules and procedures for transfer length can be used in both type of panels.

CHAPTER 3
Girder Analysis

3.1 INTRODUCTION

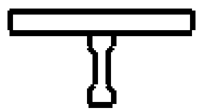
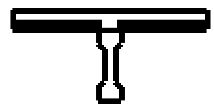
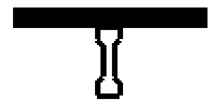
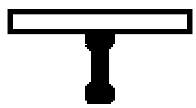
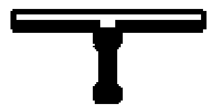
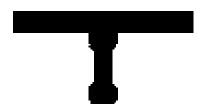
This chapter examines the results from comparative analyses of AASHTO Type IV Bridge girders designed with either normal weight or lightweight concrete. The goals of these analyses are twofold.

First, the primary focus will be to determine the possible advantages of using lightweight concrete girders in standard bridge sections. The basis for determining the advantage in this chapter will be strictly a result of comparing the hypothetical designs of the lightweight concrete girder sections with those of the identical normal weight sections. In a later chapter, estimated costs and savings due to handling, lifting, and transporting will be considered.

Second, the analyses will serve as a means to evaluate the possible use of the TxDOT prestressed girder design program for the design of lightweight concrete girders. As part of this evaluation, a procedure for using this program to design lightweight concrete girders will be recommended. This recommendation may also involve general suggestions for modifying the program to make it more compatible for designing lightweight girders. However, actual modification of the PSTRS14 program is beyond the scope of this study.

In this study, several combinations of sections utilizing both the normal and lightweight girders, as well as various normal and lightweight composite deck combinations were analyzed and compared. These girder and deck combinations are shown in Table 3.1.

Table 3.1 Girder and Deck Section Combinations Used in Analysis

	NW Deck	NW Deck/LW Panel	LW Deck
NW Beam			
LW Beam			

3.2 PSTRS14 PROGRAM

The Prestressed Concrete Beam Design/Analysis Program, commonly known as PSTRS14, was developed by TxDOT and has been in existence since 1990. According to the user guide for this program, PSTRS14 is a compilation of the essential logic and options from four TxDOT design programs, namely PSTRS10, PSTRS12, DBOXSS, AND DBOXDS [8]. These incorporated programs, in addition to some new options and logic, make PSTRS14 a versatile program that provides the user with many options for either designing or analyzing prestressed concrete girders. Because of this versatility, it will be the primary tool for designing the normal and the lightweight concrete girders in this study.

Even though PSTRS14 is a versatile program, the design of the high strength lightweight concrete girders was made cumbersome by some of the program logic that is sufficient for the design of girders made from normal weight concrete, but not for those made from lightweight. Two variables that the current program logic was unable to properly determine for the design of a lightweight girder was the modular ratio and the prestress losses. In addition, there is no means in the program to input the split tensile strength of lightweight concrete for initial cracking or shear calculations.

The modular ratio, which is used to account for the differences in stiffness between the slab and the girder, is calculated by dividing the modulus of elasticity of the concrete in the slab by the modulus of elasticity of the concrete in the girder (Ec_{slab}/Ec_{beam}). In the case of a lightweight girder with a normal weight slab, usually its modulus of elasticity will be less than the modulus of the slab, which makes the modular ratio greater than one. In comparison, the modular ratio for a normal weight girder and slab is unity or less. According to the PSTRS14 User Guide, TxDOT has historically set the modular ratio for these members equal to one if the f'c of the girder is less than 7500 psi. [8]. However, in this investigation it was deemed necessary to model the slab stiffness to be representative of the actual composite material properties. By using the actual properties, the effects of the different composite slabs on the girders can be compared as well. Because the PSTRS14 program does not set a limit on the modular ratio, it became necessary to manually limit the program so that it would obtain the proper slab section for calculation of the composite moment of inertia. This could have been accomplished with a couple of different alternatives, but the best method found was to set the modular ratio to unity by making the girder and slab modulus equal. For proper dead load deflection calculation by the program, it was determined that the modulus of the slab should be set equal to the modulus of the girder. This limiting of the modular ratio had to be done for two of the lightweight girder sections, the section with the all-normal weight deck and the section with the combined normal weight deck with lightweight panels. All other sections used their actual material properties, which will be discussed in Section 3.3, Variables Selected for Study.

From preliminary investigations using the PSTRS14 program, it was also discovered that the program would not properly calculate the prestress losses for a lightweight concrete girder. As will be discussed later in this report, prestress losses in pretensioned lightweight concrete girders are significant and can limit the effectiveness of using lightweight concrete. It has been determined from the analysis results obtained in this study that the prestress losses in lightweight concrete girders are approximately 20% higher than losses in identical normal weight girders. Further, based on the prestress loss calculations for the lightweight girder, it is known that the largest contributor of prestress loss is elastic shortening. This loss parameter is highly dependent on the initial elastic modulus (Eci). In PSTRS14 the initial elastic modulus is derived internally by applying the initial compressive strength (f'ci) and density of the girder to the AASHTO modulus equation found in Section 8.7.1 of the Standard Specification for Highway Bridges Manual. However, from previous studies of lightweight concrete, it has been determined that this AASHTO formula will overestimate the modulus of a high strength lightweight concrete girder [9]. Because elastic shortening is inversely proportional to the initial elastic modulus, the overestimated modulus will underestimate the loss due to elastic shortening. The overestimated modulus will also have an effect on the steel relaxation and concrete creep loss. Because of the inability of PSTRS14 to properly determine prestress losses, they must be determined externally and then input into the program.

As a final note about the PSTRS14 program, the program allows a user to either design or analyze a prestressed concrete girder. In designing a girder, the program determines the concrete strengths that will satisfy the given input variables. In contrast, analyzing a girder allows the user to input the concrete strengths. To maintain an equal strength basis for the different sections being analyzed, the latter method was chosen and consistently used for all analyses.

3.3 VARIABLES SELECTED FOR STUDY

The variables given in Table 3.2 are the material properties used throughout the analyses for both the normal and lightweight concrete girders. The properties for the lightweight concrete are based on testing completed for this project by Heffington, Kolozs, and Thatcher (References 4,5, and 6, respectively), while the properties for the normal weight concrete girders are derived from both tests and AASHTO code provisions. The reader should note that the lightweight concrete data, some of which was interpolated, is only representative of the mix designs developed specifically for this project. Variables for other lightweight mix designs should be developed by designers on a project specific basis.

From the data in the table, it is evident that the strengths for the normal weight concrete are exactly equal to the strengths for the lightweight concrete. This was done purposely to maintain an equal basis for comparison. The basis for these strengths was the 28-day compressive strength (f'c) and the 1-day strength (f'ci) of the lightweight concrete. Hence, the normal weight concrete strengths were assumed as equal to the strength of the lightweight concrete. The moduli of elasticity (Ec and Eci) for the normal weight concrete were determined by provisions in AASHTO 8.7.1.

The lightweight concrete girder compressive strengths (f'c) of 6000 psi and 7500 psi were established by project criteria and were the basis for mix design development by Heffington [4]. It must be noted that originally the goal was to obtain an 8000 psi mix design. However, the strengths for the 8000 psi mix design reached a plateau and sufficient confidence that this strength could be consistently obtained was not achieved. Hence, it was decided that it should be rerated as 7500 psi.

Table 3.2 Prestressed Concrete Girder Analysis Variables

	Member Type	f'c (psi)	f'ci (psi)	Ec (ksi)	Eci (ksi)
Normal Weight	Girder	7500	5500	5250	4496
	Girder	6000	4000	4696	3834
	Deck	5000	—	4287	—
	Panel	5000	—	4287	—
Lightweight	Girder	7500	5500	3390	2520
	Girder	6000	4000	3250	2435
	Deck	5000	—	2525	—
	Panel	5000	—	2525	—

The moduli of elasticity for the lightweight concrete were determined by testing. However, the sources for each of the lightweight moduli of elasticity are different. The modulus of elasticity for the 7500 psi mix design was based on testing information determined by Heffington [4], while the modulus for the 6000 psi mix design was based on consistent test measurements obtained and reported by Thatcher [6].

The material properties of the deck and panels were obtained by similar methods as the girders. That is, the moduli for the normal weight deck and panels were determined by AASHTO code provisions, while the modulus for the lightweight deck and panels were determined by the testing performed by

Thatcher [6]. The 5000 psi compressive strengths were based on strengths used in deck and panel specimens tested in this project by Kolozs [5] and Thatcher [6].

3.4 STANDARD BRIDGE SECTION FOR ANALYSIS

A bridge section that has a width and span length typical of bridges constructed in the State of Texas was selected as the basis for the analyses of all 7500 psi and 6000 psi girders discussed in this chapter. This standard section, shown in Figure 3.1, was established through discussions with the TxDOT Project Director and consists of AASHTO Type IV girders with an overall span length of 110 feet. The overall width of the section is 40 feet with girder spacing equal to 8.5 feet. The composite slab has a total depth of 8 inches and the section includes T501 railing. The T501 traffic rail is the only dead load that acts on the composite structure. This loading on the composite structure is similar to that used in the TxDOT Standard Plan Sheets for the identical section.

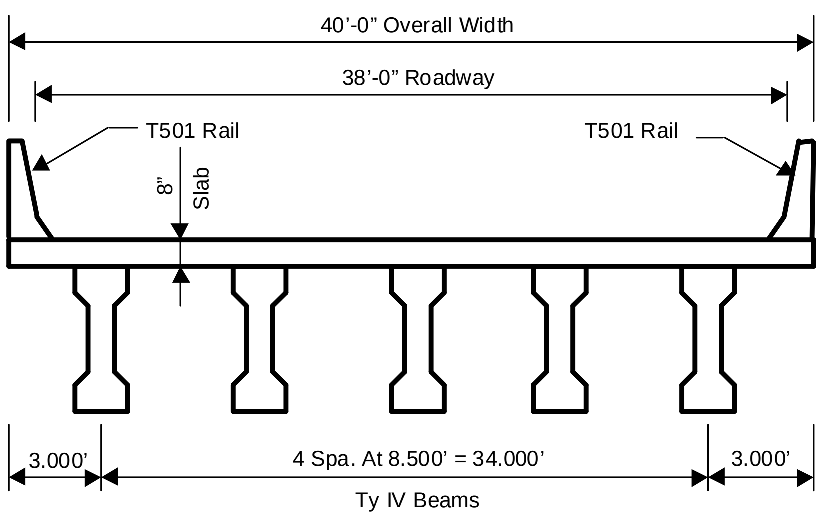


Figure 3.1 Typical Bridge Section

3.5 7500 PSI GIRDER ANALYSIS

The six section combinations, as previously shown in Table 3.1, were analyzed with 7500 psi prestressed concrete girders using PSTRS14. The results and comparison of these analyses, including input data, will be reported in the next several subsections. The focus of these analyses will be to contrast the different sections in an attempt at examining the differences between using a normal weight girder versus a lightweight girder and also at examining what differences, if any, are made by using lightweight concrete in the deck. These differences will then be discussed in Section 3.5.2, and followed by economic quantification and feasibility discussions in Chapter 4.

3.5.1 Analysis Results

The analysis results obtained in this study will be given mostly in a tabular form that has been divided into 4 separate sections. These sections include section properties; prestressing properties; flexure and shear; and camber and deflections. The tables for each section will consist of information that was either input or obtained as results (output) from PSTRS14. A distinction will be made between both types of data where appropriate.

Because of the various numbers of sections and for the purpose of easy identification, the section combinations have been represented graphically in each table in the manner illustrated in Table 3.1. The reader is reminded that normal weight concrete is identified by bordered shapes (□), whereas lightweight concrete is identified by completely solid shapes (■).

Before discussing the results, a few additional details must be clarified. The first detail involves establishment of the live loading used in the analyses. For this, the default HS20 loading in PSTRS14 was used throughout the analyses.

The next and final detail that must also be established are the allowable stresses used for design of the prestressed girder. The allowable stress criteria used for the analyses are based on AASHTO 9.15.2, with modification made to the initial allowable stress for lightweight concrete. This modification accounts for the lower modulus of rupture of lightweight concrete. From the analyses it will be evident that these stresses governed the design of the prestressed concrete girders for both concrete types analyzed.

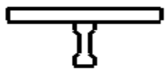
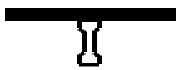
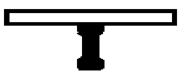
3.5.1.1 Section Properties

Section properties relate either to geometric or material properties that define the section being analyzed. Properties input into PSTRS14 are listed in Table 3.3. However, the input numerical values of these properties as well as the resulting section properties are shown in Table 3.4. Even though some of the data has been previously given in this report, Table 3.4 will provide the reader with a concise summary for supporting discussions that follow in this report.

Table 3.3 Section Properties Input into PSTRS14

Section Properties
span length
girder spacing
slab thickness
girder 28-day compressive strength (f'c)
girder 1-day compressive strength (f'ci)
slab 28-day compressive strength (f'c)
girder modulus of elasticity (Ec)
slab modulus of elasticity (Ec_{slab})
girder unit weight
slab unit weight

Table 3.4 Section Properties for 7500 psi Girders

Section	Span Length	girder Spacing	f’c girder	f’ci girder	f’c slab	Girder Unit Weight	Slab Unit Weight	Ec girder	Ec slab	n Mod. Ratio	I girder	I’ girder + deck
	(feet)	(feet)	(psi)	(psi)	(psi)	(pcf)	(pcf)	(ksi)	(ksi)		(in^4)	(in^4)
	110	8.5	7500	5500	5000	150	150	5250	4287	.817	260,403	663,174
	110	8.5	7500	5500	5000	150	134	5250	3406	.649	260,403	613,360
	110	8.5	7500	5500	5000	150	118	5250	2525	.481	260,403	552,162
	110	8.5	7500	5500	5000	122	150	3390	3390	.817	260,403	708,041
	110	8.5	7500	5500	5000	122	134	3390	3390	.817	260,403	708,041
	110	8.5	7500	5500	5000	122	118	3390	2525	.817	260,403	643,058

3.5.1.2 Prestressing Results

Before the results from the prestressing of the girders are presented, it is appropriate to identify the PSTRS14 inputs for the girder. These inputs that are required for defining prestressing include pretensioning strand properties and layout, as well as prestress losses, allowable tension coefficients, and stress due to external loads. Table 3.5 is a summary of the prestressing properties used for the analyses. These material properties were kept constant for the analytical study of the 7500 psi girders.

Table 3.5 Prestressing Variables Input into PSTRS14 for 7500 psi Girders

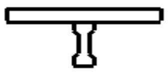
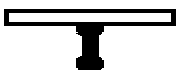
Prestressing Variables	
Variable	**Value**
No. of Strands	Varies, see Table 3.6
Strand Eccentricty (Center)	Varies, see Table 3.6
Strand Eccentricity (End)	Varies, see Table 3.6
Strand Size	½ inch
Strand Type	7-Wire Lo-Rlx
Strand Area	0.153 sq. inches
Strand Ultimate Strength	270 ksi
Es	28000 ksi
No. of Straight Web Strands	0
No. of Web Strands/Row	2
Relative Humidity	50 percent
Dist. CL to Hold Down	5.42 feet

Prestress losses shown for the normal weight girders were calculated internally by the PSTRS14 program, whereas, the losses for the lightweight girders were calculated externally and input into the program. The only other values required for the prestressed girder analyses were values for "stresses due to the total external load at centerline", top and bottom, and the initial allowable tension coefficient. The values for the stresses due to external loads varied and were determined on a case by case basis then input into PSTRS14.

The initial allowable tension coefficient was modified from the default 7.5 to 6.3 for the lightweight concrete. This is in accordance with AASHTO 9.15.2.3, which suggests that modulus of rupture for sand-lightweight concrete is equal to 6.3 times the square root of the 28-day compressive strength. Even though modification was made to the initial allowable tension coefficient, the final allowable tension coefficient for the lightweight girder was not modified for these analyses. This is because the default coefficient used in PSTRS14 for final allowable stresses is approximately 5 percent lower than the 6.3 times the square root of the 28-day compressive strength recommended by AASHTO. In retrospect, until a better understanding of the allowable stresses for lightweight concrete can be established, it is advisable to provide a larger margin of safety by lowering the coefficient even further. The impact of this should be minimal. As an example, lowering the final tensile coefficient to 5.0 for the 7500 psi all-lightweight concrete section would require the addition of only two more prestressing strands. The addition of these two strands would satisfy this lower allowable stress.

The prestressing results from the PSTRS14 analysis for each of the section combinations of the 7500 psi girders are given in Table 3.6. Also, Figure 3.2 was prepared to show the general pretensioned strand arrangement for the girders.

Table 3.6 Prestressing Results for 7500 psi Girders

Sections	No. of Strands	Strand Eccentricity		Prestress Losses		Stress due to Tot. External Load		Stress @ End (Release)		Stress @ CL (Final)	
		End	CL	Release	Final	Top	Bott	Top	Bott	Top	Bott
		(in)	(in)	(percent)	(percent)	(psi)	(psi)	(psi)	(psi)	(psi)	(psi)
	50	11.07	19.47	8.88	26.02	3768	-4087	35	3276	2715	-512
	48	10.92	19.67	8.55	25.40	3689	-3973	57	3136	2645	-492
	46	12.23	19.88	8.23	24.82	3658	-3878	-138	3179	2624	-494
	50	11.07	19.47	14.92	31.43	3384	-3798	33	3059	2408	-485
	46	12.23	19.88	13.94	29.72	3204	-3646	-129	2981	2238	-483
	44	12.02	20.02	13.38	28.94	3142	-3539	-97	2847	2192	-467

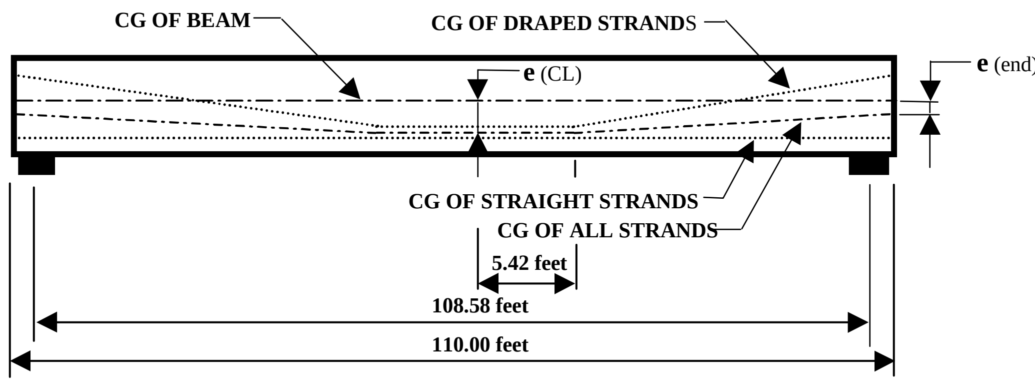


Figure 3.2 Prestressed Girder Strand Layout

3.5.1.3 Flexure and Shear Results

Using the analyze option in PSTRS14 necessitates that the "Ultimate Moment Required" be input into the PSTRS14 program. The ultimate moment required consists of the moment due to dead loads acting on the girder, including the girder self-weight, as well as due to AASHTO HS20 live load. The program then determines, based on geometric properties of the composite section, strand centroid, material properties of pretensioning steel and concrete, the "Ultimate Moment Provided." The program also determines 1.2 x Mcr, Mcr being the cracking moment, and compares it to the "Ultimate Moment Required". The results for the flexure and shear calculations as determined by PSTRS14 are shown in Table 3.7. Note that for all girders, the ultimate moment provided far exceeds the ultimate moment required. This indicates that the allowable stresses governed the design of the prestressing.

3.5.1.4 Camber and Deflection

Cambers and deflections for the 7500 psi girders are tabulated in Table 3.8. According to the User Guide for PSTRS14, camber is determined based upon the hyperbolic function method developed by Sinno [10]. However, the Guide also goes on to say that any value predicted is only an estimate because of the many factors influencing this variable. Nevertheless, it is obvious from the analyses results that the camber for the lightweight girders is higher than the camber for normal weight girders.

Instantaneous elastic dead load deflections for the lightweight concrete girders are higher than that for the normal weight girders. This is a result of the lower modulus of elasticity characteristic in lightweight concrete. Comparisons of the modulus, camber, and deflections will be discussed in subsequent sections. As a final note, deflections determined by PSTRS14 are based on the dead load of the slab and rail in these analyses.

In summary, variables representative of both the normal and lightweight concrete designs used in the analyses were predetermined and were based on testing or AASHTO code provisions. These analyses were performed using the predetermined variables and TxDOT's PSTRS14 program for designing prestressed concrete girders. The important thing to note about the 7500 psi mix was that a design could be achieved for each of the different section combinations at the predetermined span length and girder spacing.

Table 3.7 Flexure and Shear Analysis Results for 7500 psi Girders

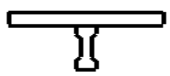
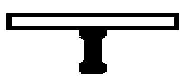
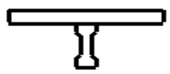
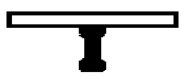
Sections	Shear Stirrup Spacing		Ultimate Horiz. Shear Stress	Ultimate Moment Required	Ultimate Moment Provided
	Near End	Near CL			
	(in)	(in)	(psi)	(k-ft)	(k-ft)
	12	12	236.2	6862	9033
	12	12	218.5	6688	8731
	12	12	195.7	6514	8427
	12	12	235.5	6568	9033
	12	12	229.4	6394	8427
	12	12	210.5	6221	8107

Table 3.8 Cambers and Deflections for 7500 psi Girders

Sections	Maximum Camber	Dead Load Deflections (Centerline)		
		Slab	Other	Total
	(ft)	(ft)	(ft)	(ft)
	.301	-.162	-.010	-.172
	.283	-.145	-.011	-.155
	.278	-.128	-.012	-.139
	.419	-.251	-.014	-.265
	.393	-.224	-.014	-.238
	.369	-.197	-.016	-.213

3.5.2 Discussion of Analysis

With the reporting of the analyses results, attention can now be focused on contrasting major differences between the two designs with normal and lightweight girders to gain an understanding of advantages and disadvantages. The comparison will begin by examining prestressing conditions followed by a look at strength and serviceability results.

3.5.2.1 Prestress Losses

The material property that makes lightweight concrete an appealing alternative to normal weight concrete is its low density. In the case of the mix designs developed for this study, the density of the lightweight concrete is approximately 30 pcf less than a normal weight concrete with the same strength. This represents approximately a 20 percent reduction in dead load due to self-weight. However, accompanying the lower density is a lower modulus of elasticity for the lightweight concrete. From Table 3.2, the modulus of the lightweight 7500 psi girder is 3390 ksi compared to 5250 ksi for the normal weight girder. This indicates that the elastic modulus for the lightweight concrete is approximately 65 percent that of normal weight concrete. The lower modulus of this material results in much higher initial elastic loss in prestress. This counteracts the benefits of the lower density, especially in a single stage pretensioning application. Evidence of this can be noted in the predicted prestress losses for the girders given in Table 3.6 for which the losses are dependent upon the initial elastic modulus.

The higher prestress losses in lightweight concrete are also evident in comparing the normal weight and lightweight girders with normal weight decks. It is interesting to note that both require an equal number of prestressing strands. Intuitively, one would think that the lightweight girder would require fewer strands due to its lower density of 122 pcf. However, as will be shown, higher prestress losses in this material counteract the dead load reduction and hence reduce the potential for material savings.

To show the importance of the prestress losses for the lightweight girders, Figure 3.3 was developed. This figure depicts the variation of initial and final prestress losses as the number of prestressing strands for the normal and lightweight sections described above are varied between 40 and 60 strands.

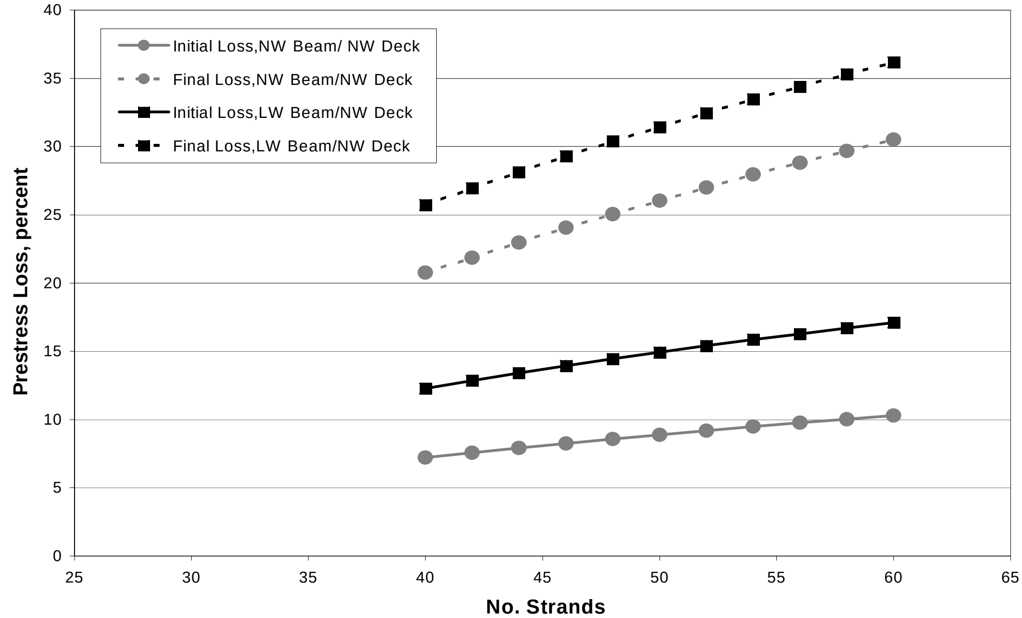


Figure 3.3: Comparison of Prestress Losses

From this figure, it is evident that the prestress losses are considerably higher for the lightweight girder. On average, the initial losses are approximately 68% higher for the lightweight as compared to the normal weight, while the final losses are approximately 21% higher. The higher losses for the lightweight concrete girder can be attributed mostly to the lower modulus of elasticity that is typical of the lightweight concrete.

The higher prestress losses determined for the lightweight concrete girder translate directly to a lower effective prestress force for this member as determined by Equation 3.1 and as shown in Figure 3.4. This is not surprising considering the fact that the effective prestress force is directly proportional to the loss of prestress.

$$.75 \times f'_s \times A^*_s \times N \times (1 - \Delta f_s) \qquad \text{(Equation 3.1)}$$

In Equation 3.1, f'_s equals the ultimate stress of prestressing steel; A^*_s equals the area of prestressing steel; N is the number of prestressing strands; and Δf_s is represents the total prestress loss, excluding friction.

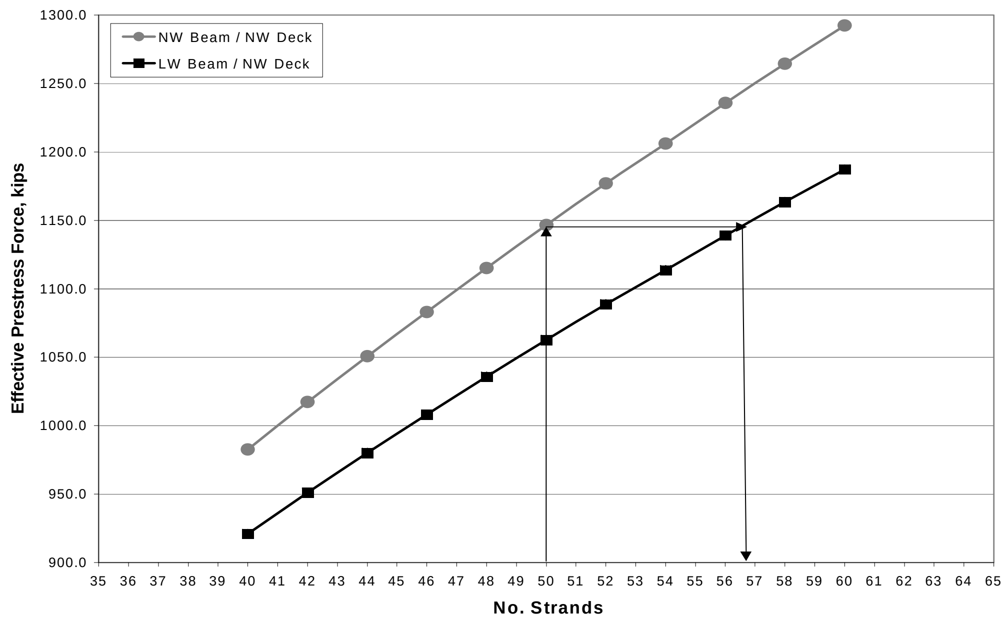


Figure 3.4: Comparison of Effective Prestress Force

This figure can also be used to determine the number of strands that would be required for a lightweight girder to maintain the same prestress force as a girder made from normal weight concrete. As an example, if 50 strands were required for a normal weight girder with a normal weight deck, approximately 58 strands, rounding up to next even increment would be required for the lightweight section (shown by lines with arrows). This difference is of course again due to the higher prestress losses for the lightweight girder. This difference in strand requirements however does not directly explain why the sections being compared both require 50 strands. To further examine why the same numbers of strands are required, a comparison of the effective stresses for both girders that include the self-weight of the members is necessary.

Figures 3.5 and 3.6, present the comparisons of effective stress for the normal and lightweight girder. Figure 3.5 shows the effective stress from the prestress force and really does not offer any new information. However, Figure 3.6 is the effective stress taking into account the stress induced by the self-weight of the girders. From this figure, it is evident that the curves for the effective stress of these two girders become almost coincident with each other. This indicates that the difference in prestress losses in combination with the difference in self-weight cause these two members to experience almost the same stress, and this is almost exactly the case if the members each have 50 strands.

The final interpretation of this is that if the total external superimposed loads for each of the sections are considered equal, then the effective stress including self-weight for these sections are nearly equal. This is a result of the difference in prestress losses as well as the differences in stress due to the density of the concrete used in the girders. This leads to the conclusion that the overall benefit of the lightweight section due to the lower density can be considered in this case to be negligible when compared to a similar normal weight section. This highlights the fact that the prestress losses play a very large role in the effectiveness and hence the efficiency of a girder made from lightweight concrete.

Prestress losses in this analysis were determined using the AASHTO method. However, because these losses in the lightweight concrete are crucial to the efficiency of the lightweight girder, the ACI-ASCE Committee 423 method was used as a comparison check. This comparison is shown in Table 3.9. It can be noted that the prestress losses determined by each method are more similar for the lightweight concrete than for the normal weight concrete. However, it must be considered that the differences would be much more pronounced if the maximum limits suggested by the ACI-ASCE Committee 423 method were used in the calculations. These maximum limits are 40,000 psi for normal weight concrete and 45,000 psi for lightweight concrete. It must also be re-emphasized that the greatest difference in the losses between the normal and lightweight concrete is due to elastic shortening which is inversely proportional to the initial modulus of elasticity.

Table 3.9: Comparison of Prestress Loss Methods

	Normal Weight		Lightweight	
	AASHTO (psi)	ACI-ASCE (psi)	AASHTO (psi)	ACI-ASCE (psi)
Shrinkage	9,500.0	8,220.0	9,500.0	8,220.0
Elastic Shortening	17,195.2	16,890.0	30,049.6	32,390.0
Creep	24,423.2	15,110.0	23,759.4	21,340.0
Steel Relaxation	1,584.3	3,390.0	332.1	2,660.0
Total:	52,702.7	43,610.0	63,641.1	64,610.0
% Difference in Totals	+21%		-1.5%	

Initial Prestress Loss	26.03%	21.50%	31.43%	30.20%
Final Prestress Loss	8.88%	9.18%	14.92%	16.70%

Note: Initial Prestress Loss was taken as ES + .5 CRs
Max loss for normal weight concrete of 40,000 psi
Max loss for lightweight concrete of 45,000 psi

From the examination of the two equivalent sections, with one section consisting of a normal weight girder and the other a lightweight girder and both with normal weight decks, it has been shown that the higher prestress losses for girders made from lightweight concrete reduce the girder's overall effectiveness. This causes the total effective stress (including self-weight of the girders) to be almost identical for these girders, hence the lightweight girder for this scenario does not appear to have an advantage over a girder made from normal weight concrete.

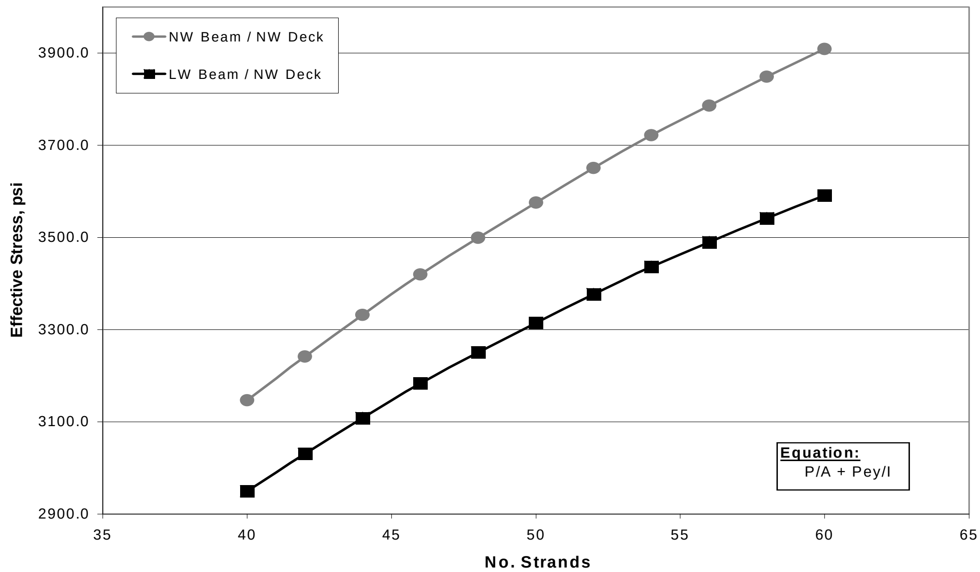


Figure 3.5: Effective Stress at Bottom Centerline of Girder

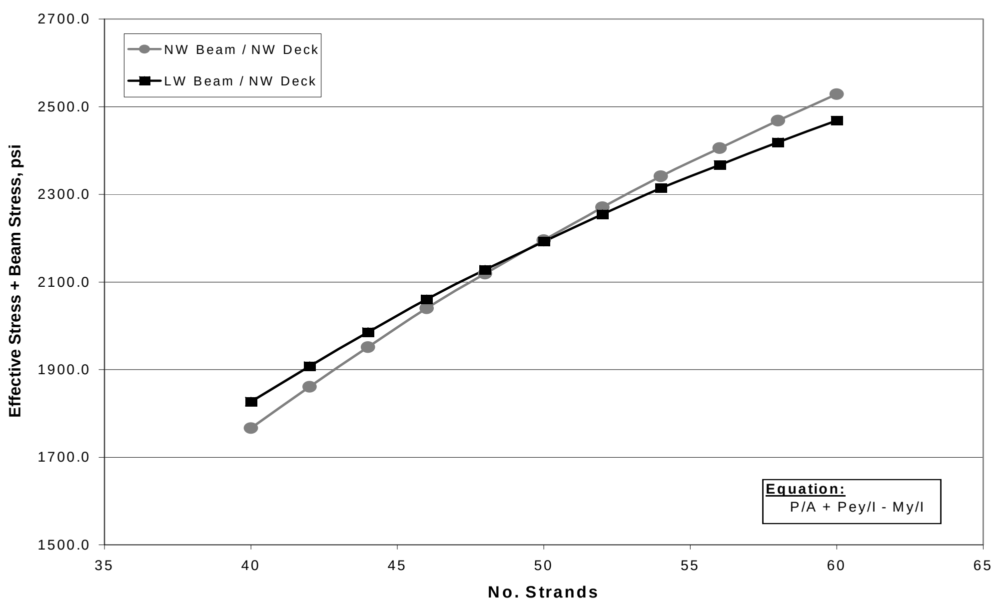


Figure 3.6: Effective Stress plus Self-Weight at Bottom Centerline of Girder

A possible alternative to overcoming the elastic shortening losses that are crucial due to the low initial elastic modulus of the lightweight concrete would be a post-tensioned application. In post-tensioning, the elastic losses occur prior to anchoring the tendon and thus are replaced by the much lower anchor set. In addition, the girder will have a higher f'ci at stressing since the concrete usually has much more maturity that results in a higher Ec and lower losses. Post-tensioned applications would take greater advantage of this material's low density and offer larger potential for material savings.

Continuing with the examination of the prestress losses, Figure 3.7 presents a graphical look at the differences in initial and final prestress losses between all the different sections. From this bar graph, it is obvious that the losses, both initial and final, are higher for the lightweight concrete. Again, this can be attributed to the lower modulus of elasticity. However, this graph also shows a consistent trend that indicates that with increasing amounts of lightweight concrete used within a section, lower prestress losses will result. This trend that can be seen with either the normal weight or lightweight sections is due to the fact that with increasing the amounts of lightweight in a section, a reduction of the number of prestressing strands is possible (see Figure 3.8) and this in turn reduces the prestress losses. Figure 3.8, also indicates that approximately a 12 percent savings in strands can be realized between an all normal weight section and an all lightweight section. In a very large bridge, this savings could add up to be substantial.

As a final note, it can be said that the design of the girders was governed by AASHTO stress limitations instead of by strength provisions. A look at the final stresses induced in the girder section shown in Figure 3.9 reveals that compressive stresses at the centerline are approximately 20 percent lower for the all lightweight section compared to the all normal weight section.

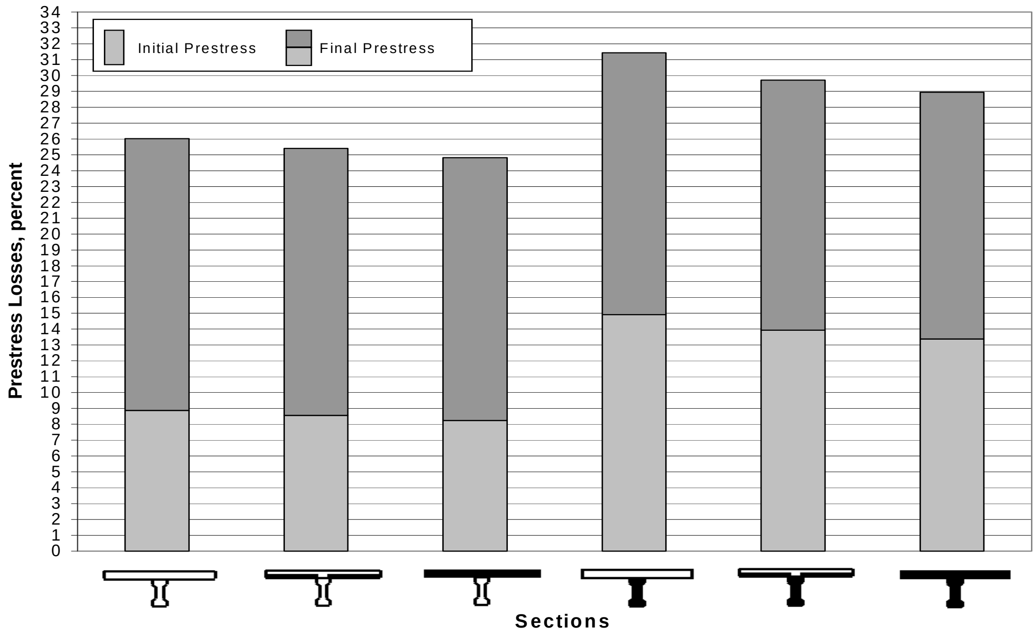


Figure 3.7 Initial and Final Prestress Losses for 7500 psi Girders

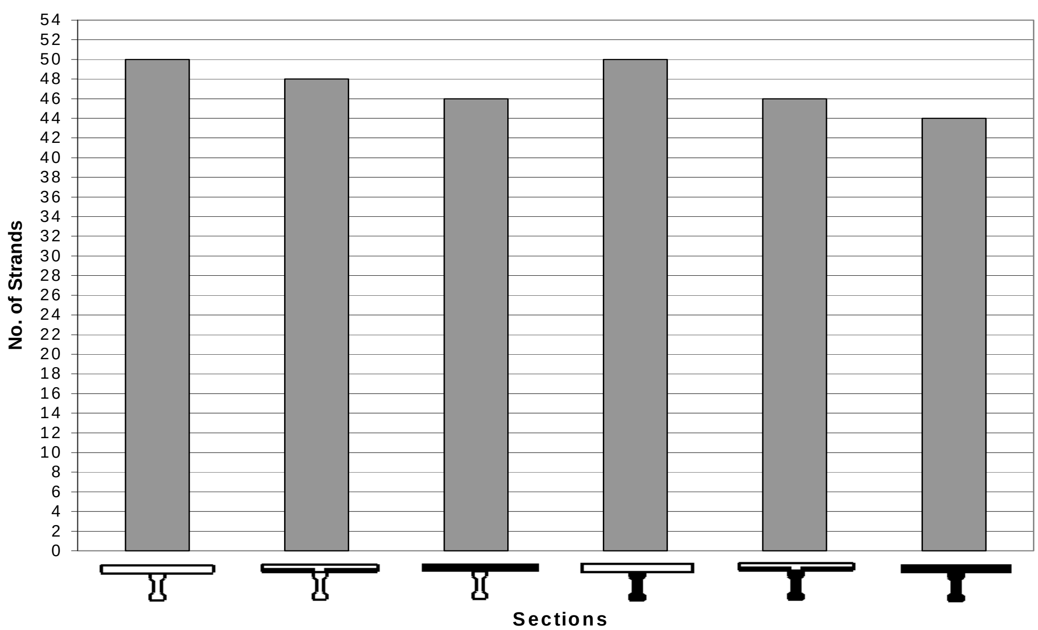


Figure 3.8 Prestressing Strand Requirements for 7500 psi Girders

This correlates well with the reduced density, which was previously mentioned to equal approximately this same amount. Examination of the tensile stresses at the centerline for each of the section reveals that there is essentially no difference. However, this can be expected because the tensile stress at the centerline usually controls the design of a prestressed girder and the fact that each girder was optimized to have the least prestressing strands possible.

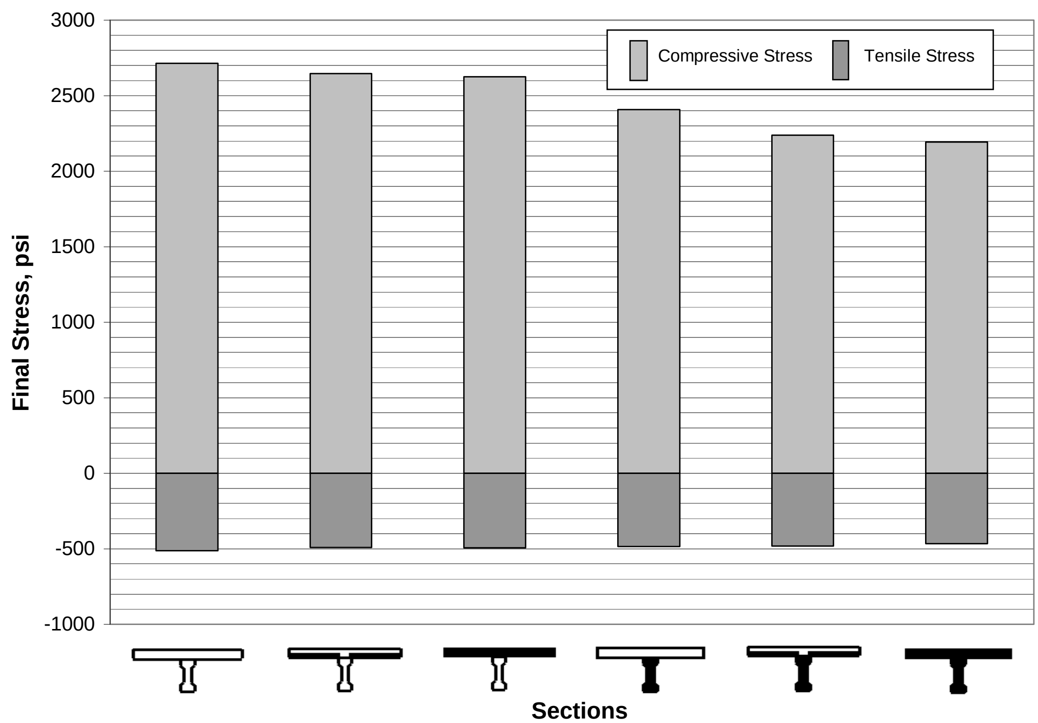


Figure 3.9 Final Compressive and Tensile Stresses at Centerline for 7500 psi Girders

In comparison to the high tensile stresses that typically controlled at the centerline, final tensile stresses at the end were less than 25 percent those at the centerline. These end tensile stresses were not a factor and did not control.

3.5.2.2 Flexure and Shear

From the shear results obtained by PSTRS14 and given in Table 3.7, the shear stirrup spacing indicates that there is no difference in web reinforcing spacing for the six different sections. However, this is not taking into account the splitting tensile strength of the lightweight concrete that will more than likely require closer stirrup spacings.

Considering flexure, Table 3.7 shows that a 10 percent difference in moment required exists between the all-lightweight section and the all-normal weight section. Note that only a 10 percent reduction in moment is obtained even though there is a 20 percent reduction in dead load. This can be rationalized by the fact that the moments due to factored dead load represent only 50 percent of the total moment. The other 50 percent is made up of the factored live load moment.

Figure 3.10 depicts both the required and provided ultimate moments for each of the sections. From this figure and Table 3.7, it can be noted that the provided ultimate moment exceeds the required moment by approximately 30 percent.

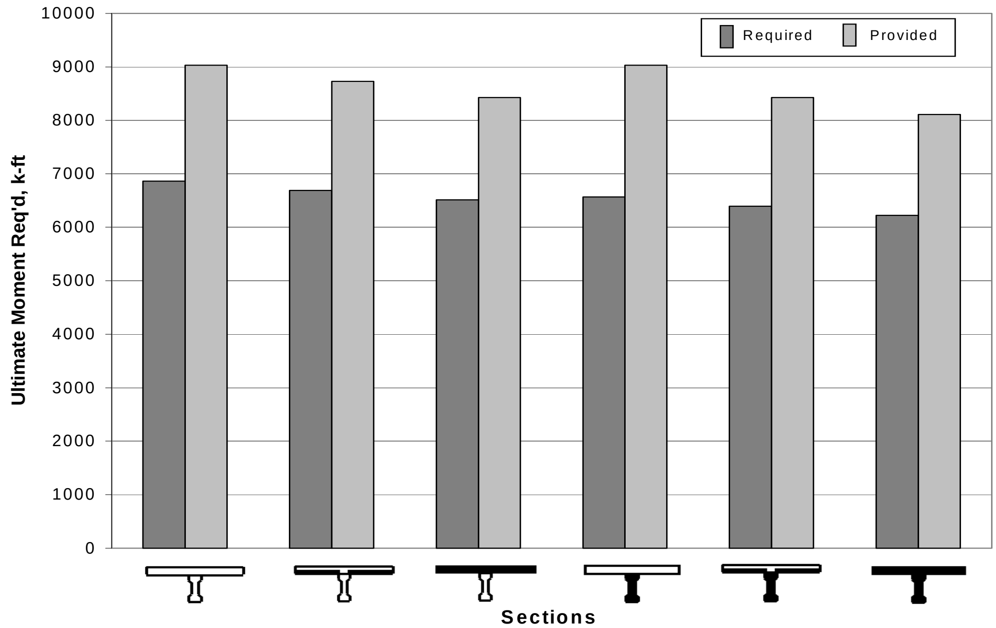


Figure 3.10 Ultimate Moment Required and Provided for 7500 psi Girders

3.5.2.3 Camber and Deflection

A look at camber and instantaneous elastic deflections due to dead load shown in Figure 3.11 and Figure 3.12, respectively, shows that the lightweight concrete girders are more flexible than the normal weight girders. This is due to the lower modulus of elasticity of the lightweight concrete. Comparing the average deflections of both the normal weight and lightweight girders shows that the dead load deflections for the lightweight girders average approximately 2.9 inches, whereas the deflection for the normal weight girders averages 1.9 inches. This is a 50 percent increase in deflections for the lightweight girders. For the camber, the average camber of the lightweight girder is approximately 4.8 inches, which represents a 40 percent increase over the 3.4 inch average camber for the normal weight girders.

Computation of the net deflection with a critical value of one inch indicates that all of the sections will satisfy this condition. Net deflection is computed by subtracting eight-tenths the slab dead load deflection from the camber. Results for each of the sections is given in Figure 3.13.

Given the above information, it must be emphasized that the camber calculations simply represent an estimate. It will be interesting to see at erection how the actual cambers will compare to those predicted by PSTRS14, but even at this point it may difficult to establish how these girders will behave. This is because there are so many variables that influence the camber. Also, consider that because camber and deflection are the difference of two large numbers, at first glance, a difference of 100 percent between a field measurement and the predicted camber value may actually only be a difference of very small magnitude. What should actually be considered is the difference (measurement to predicted) over the sum of the absolute values of the camber and deflection.

Another variable that may certainly effect the camber is the prestress loss. If the loss has been overestimated then the camber may actually be larger than predicted, and vice versa if the loss is underestimated. It is advisable with the uncertainty of the cambers for the lightweight girders that the contractor closely monitor these in the field. With close monitoring, the contractor will have the opportunity to make any cap and bearing seat elevation adjustments which may be necessary.

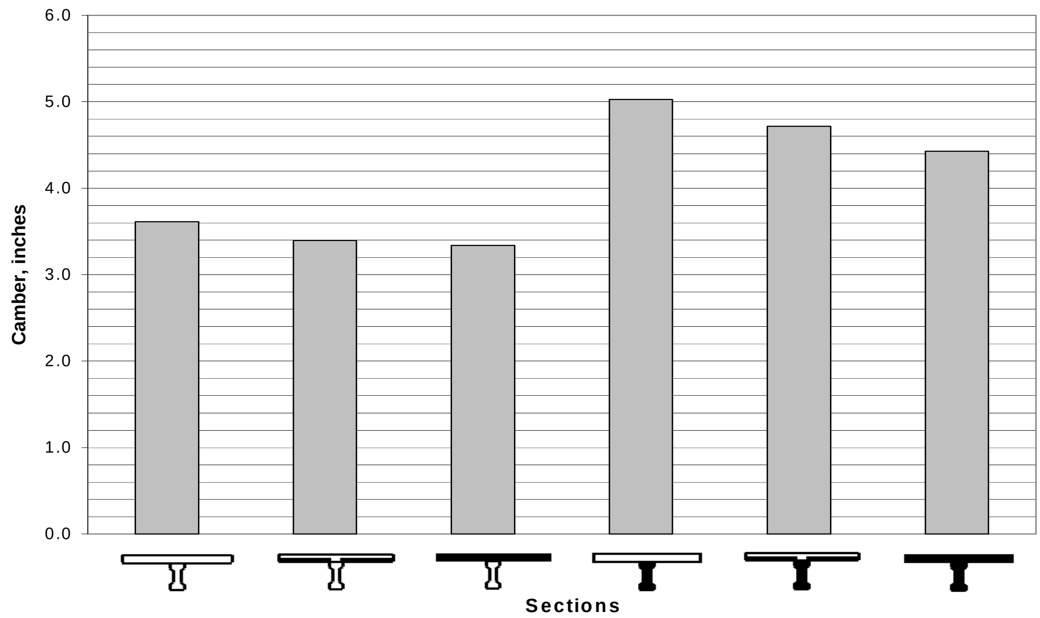


Figure 3.11 Camber for 7500 psi Girders

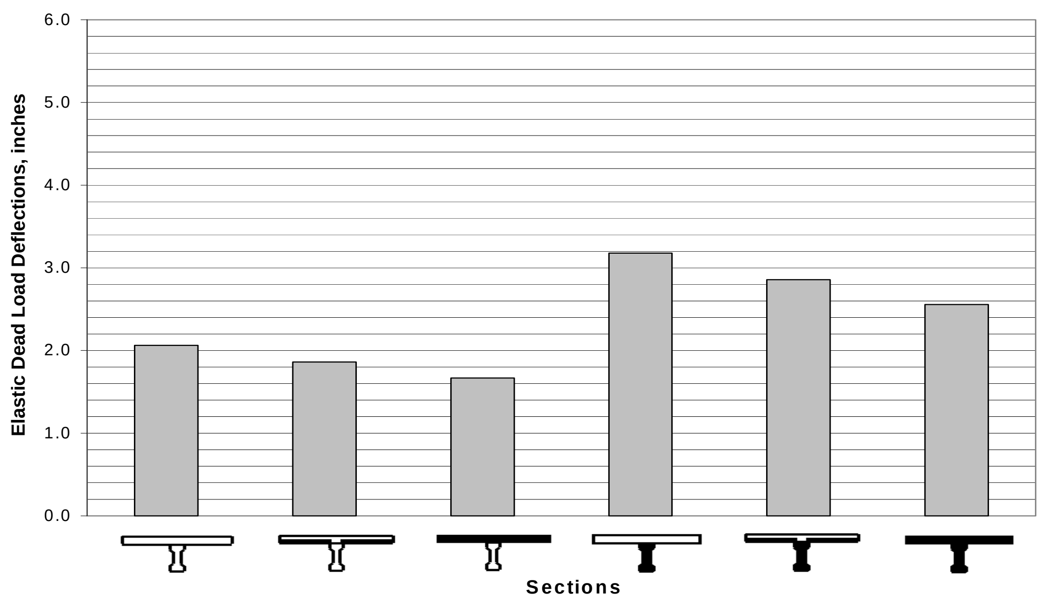


Figure 3.12 Elastic Deflections due to Dead Load for 7500 psi Girders

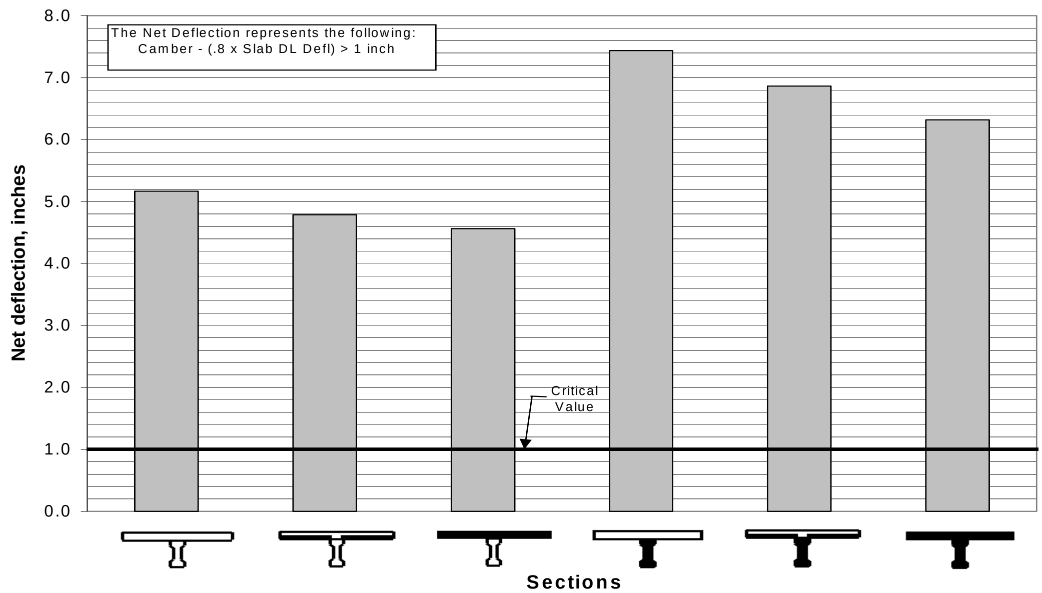


Figure 3.13 Net Deflection for 7500 psi Girders

3.6 6000 PSI GIRDER ANALYSIS

In the previous section, the results from the 7500 psi concrete analyses were detailed and discussed. From the analyses, it was shown that the 7500 psi girders could satisfy the predetermined span and girder spacing. The preliminary analyses of the 6000 psi girders indicates that the initial strength of this concrete (f'ci), equal to 4000 psi, is too low for release of the prestressed girder at one day. This is because the initial allowable concrete stresses cannot be satisfied for this girder at the span length and girder spacing chosen for the model. Because of this, two different approaches were taken to analyze and compare the 6000 psi concrete girders.

The first approach was to maintain the span of 110 feet and to determine the initial concrete strength that would be necessary for the 6000 psi girders to satisfy the allowable stress criteria as given in AASHTO. The second approach was to maintain the initial compressive strength of 4000 psi (strength actually determined by testing of mix) and then to figure what maximum span length could be achieved based on this initial strength.

The two approaches will be presented in the following sections, beginning with the analyses of the sections with constant span lengths and followed with the analyses of sections with constant initial strength. True comparisons of the 6000 psi sections will be difficult because of the varying properties. However, these analyses will provide information that will be beneficial in determining the type of section most suitable for a particular application.

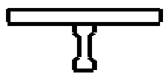
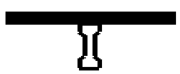
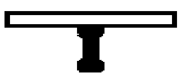
3.6.1 Analysis Results for Constant Span Length

3.6.1.1 Section Properties

The section properties for the 6000 psi girders with a constant span length will be given in this section. These properties are given in Table 3.10. The same general format, as well as the same properties, previously given for the 7500 psi girders will again be presented for the 6000 psi girders. The variables used as input into the PSTRS14 program are given in Table 3.3.

Obviously, for reasons already mentioned, one of the variables most effected by the change in mix designs from 7500 psi to 6000 psi was the concrete initial strength. The reason for highlighting this variable is due to the fact that a concrete precaster relies on this initial strength for determining when the precast girders can be released. As reported by Heffington, the precast manufacturer that made the girders for the testing in this project required that a minimum strength of 3500 psi be achieved before release of the girders.

Table 3.10 Section Properties for 6000 psi Girders with Constant Span Length

Section	Max. Span Length	Girder Spacing	f’c girder	f’ci girder req’d	f’c slab	Girder Unit Weight	Slab Unit Weight	Ec girder	Ec slab	n Mod. Ratio	I girder	I’ girder + deck
	(feet)	(feet)	(psi)	(psi)	(psi)	(pcf)	(pcf)	(ksi)	(ksi)		(in^4)	(in^4)
	110	8.5	6000	5700	5000	150	150	4696	4287	.913	260,403	687,810
	110	8.5	6000	5500	5000	150	134	4696	3406	.725	260,403	637,288
	110	8.5	6000	5300	5000	150	118	4696	2525	.538	260,403	574,364
	110	8.5	6000	5300	5000	118	150	3250	3250	1.00	260,403	708,041
	110	8.5	6000	4900	5000	118	134	3250	3250	1.00	260,403	708,041
	110	8.5	6000	4825	5000	118	118	3250	2525	.777	260,403	652,257

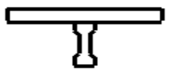
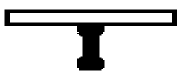
3.6.1.2 Prestressing Results

The prestressing strand properties for the analyses of the 6000 psi girders are given in Table 3.11. The same strand was used in both the design of the 7500 psi and 6000 psi girders The constant variables given in this table are identical to those used for the analyses of the 7500 psi girders. However, the variables that are not constant and which are unique to the 6000 psi (constant span length) girder design are given in Table 3.12.

Table 3.11 Prestressing Variables Input into PSTRS14 for 6000 psi Girders

Prestressing Variables	
Variable	**Value**
No. of Strands	Varies, see Table 3.12
Strand Eccentricty (Center)	Varies, see Table 3.12
Strand Eccentricity (End)	Varies, see Table 3.12
Strand Size	½ inch
Strand Type	7-Wire Lo-Rlx
Strand Area	0.153 sq. inches
Strand Ultimate Strength	270 ksi
Es	28000 ksi
No. of Straight Web Strands	0
No. of Web Strands/Row	2
Relative Humidity	50 percent
Dist. CL to Hold Down	5.42 feet

Table 3.12 Prestressing Results for 6000 psi Girders with Constant Span Length

Sections	No. of Strands	Strand Eccentricity		Prestress Losses		Stress due to Tot. External Load		Stress @ End (Release)		Stress @ CL (Final)	
		End	CL	Release	Final	Top	Bott	Top	Bott	Top	Bott
		(in)	(in)	(percent)	(percent)	(psi)	(psi)	(psi)	(psi)	(psi)	(psi)
	52	11.21	19.29	9.05	26.90	3723	-4069	13	3420	2665	-416
	50	11.07	19.47	8.87	26.41	3638	-3954	35	3276	2591	-398
	48	10.92	19.67	8.67	25.88	3600	-3856	57	3132	2563	-397
	52	11.21	19.29	15.92	32.98	3341	-3762	12	3162	2371	-413
	48	10.92	19.67	14.96	31.34	3161	-3609	53	2916	2200	-405
	44	12.02	20.02	13.87	29.45	3080	-3495	-97	2831	2137	-445

3.6.1.3 Flexure and Shear Results

For the analysis of the 6000 psi girders in flexure, the "Ultimate Moment Required" was needed for input into the PSTRS14 program. This input variable consists of the moments due to dead loads acting on the girder, including the girder self-weight, as well as moment due to the AASHTO HS20 live load. Based on geometric properties of the composite section, the strand centroid, material properties of pretensioning steel, and concrete the program internally determines the "Ultimate Moment Provided." A comparison of the required moment and the provided moment is then made. In addition, the program determines the value for 1.2 x Mcr and also compares it to the "Ultimate Moment Required". For the analysis of 6000 psi girders with constant span length, 1.2 x Mcr never governed. Table 3.13 contains the results for the flexure and shear calculations as determined by PSTRS14. Note that for all girders, the ultimate moment provided far exceeds the ultimate moment required. This indicates that the allowable stresses governed the design of the prestressing.

3.6.1.4 Camber and Deflection

Cambers and instantaneous elastic deflections due to dead load for the 6000 psi girders are tabulated in Table 3.14. The results for both these variables were determined by PSTRS14. Camber, which is due to the eccentricity of the pretensioning strands, is determined within the PSTRS14 program by a hyperbolic function method developed by Sinno [10]. Even though the method for determining the value of the camber may be based on some complex function, the results obtained are simply an estimate. Camber is influenced by many variables and it is difficult if not impossible to obtain an exact solution. The high degree of variability is confirmed by findings in Research Report 381-1 by Kelly, Bradberry, and Breen [11]. In this report, it is noted that for a Type IV long span girder the midspan camber at erection and at final camber can vary by more than 3 inches with typical conditions. The variable that is going to distinguish the values of camber and deflection between the normal weight and lightweight girders is the modulus of elasticity.

Table 3.13 Flexure and Shear Analysis Results for 6000 psi Girder with Constant Span Length

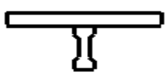
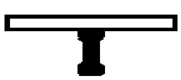
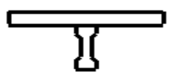
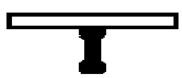
Sections	Shear Stirrup Spacing		Ultimate Horiz. Shear Stress	Ultimate Moment Required	Ultimate Moment Provided
	Near End	Near CL			
	(in)	(in)	(psi)	(k-ft)	(k-ft)
	12	12	241.5	6862	9332
	12	12	224.4	6688	9033
	12	12	202.4	6514	8731
	12	12	234.0	6526	9332
	12	12	228.0	6352	8731
	12	12	211.1	6179	8107

Table 3.14 Cambers and Deflections for 6000 psi Girders with Constant Span Length

Sections	Maximum Camber	Dead Load Deflections (Centerline)		
		Slab	Other	Total
	(ft)	(ft)	(ft)	(ft)
	.332	-.181	-.011	-.192
	.314	-.162	-.011	-.173
	.295	-.143	-.013	-.155
	.456	-.262	-.015	-.277
	.415	-.234	-.015	-.249
	.386	-.206	-.016	-.222

The analyses for the 6000 psi girders were based on predetermined variables and cross sectional properties. The results presented in this section are for the normal and lightweight concrete mixes in which a constant span length of 110 feet was maintained. This resulted in variable initial compressive strengths (f'ci) required for all the different composite sections. Differences in these initial strengths and in other variables will be examined in the section that follows.

3.6.2 Discussion of Analysis

Preliminary analyses determined that the 6000 psi girders with an initial strength (f'ci) of 4000 psi did not satisfy the allowable stress provisions in AASHTO 9.15.2. Because of this, the analysis initially focused on determining what initial strength would be required for each of the sections to satisfy these allowable stresses. The analyses were completed by iterating between the concrete strengths, the number of prestressing strands, and the prestress losses.

With the results from the analyses of the 6000 psi concrete girders with a constant span length given, comparisons between the normal and lightweight concrete designs will be made. The discussion of these results will begin with section properties and will continue with some of the more important differences of prestressing, flexure, shear, camber, and deflections.

3.6.2.1 Section Properties

As shown in Figure 3.14, the minimum f'ci that satisfies allowable stress criteria for the various sections ranges from a high of 5700 psi for the all normal weight section to a low of approximately 4800 psi for the all lightweight section. This represents about a 20 percent difference in initial strengths required between the extreme sections. Also from this figure, the general trend between the various sections indicates that the required initial strength decreases with an increase in the amount of lightweight concrete, with indistinguishable difference between a normal weight girder with a lightweight deck and a lightweight girder with normal weight deck. The reason for this indistinguishable difference is due to the fact that the controlling compressive end stresses at release for both these members are almost identical. Further, the densities of both these sections per unit length are also very similar. This will have a bearing on the total number of strands in each of the girders and consequently, the lightweight girder section requires a total of 4 more strands than the normal girder section. This is due to the higher losses associated with the lightweight concrete.

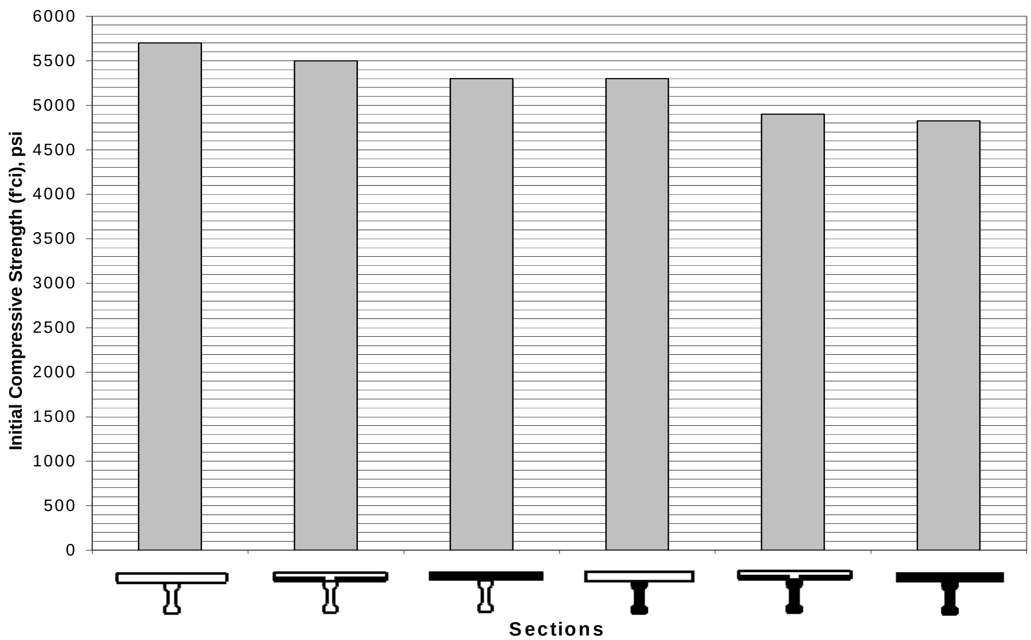


Figure 3.14 Initial Compressive Strength Requirements for 6000 psi Girders with Constant Span Length

3.6.2.2 Prestressing

The importance of initial strength in terms of release of the pretensioning strands has already been noted. At this point, it is important to see what the relationship is between these required initial strengths and the original strength gain curve. The original strength gains were determined from testing by Heffington [4] and is shown in Figure 3.15. From this plot, it is evident that the range of days needed for curing the 6000 psi lightweight girder so that it can achieve the required strengths is between 2 and 3 days. These results assume that the components of the lightweight concrete mix design are held constant, hence the mix design is unaltered. For the normal weight concrete, without a specific mix design the number of days needed for curing cannot be accurately established. However, this is not a concern for normal weight designs since plants have these types of mix designs that they regularly use and that achieve the required strengths.

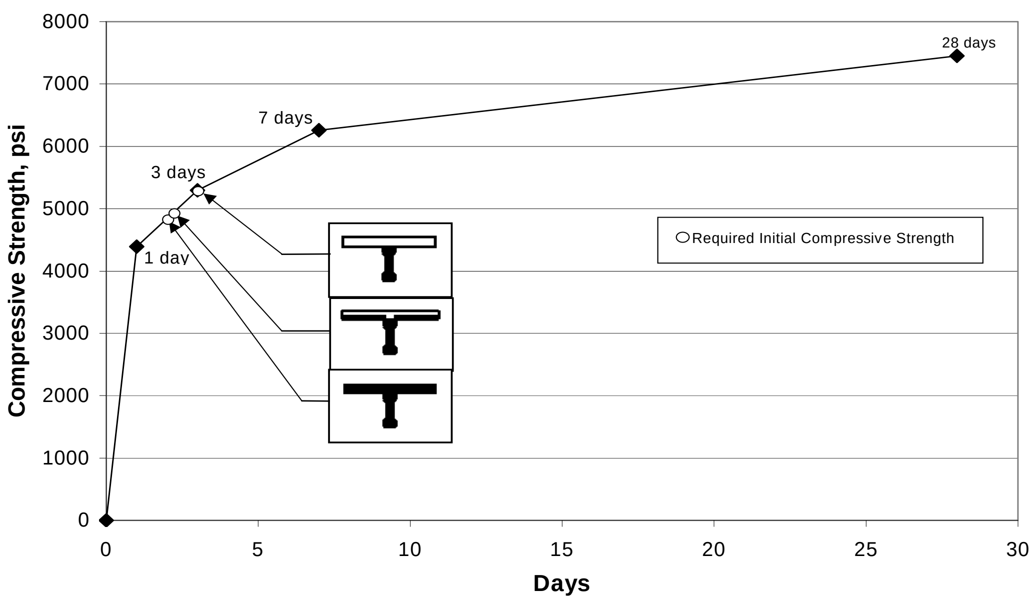


Figure 3.15 Compressive Strength Gain for 6000 psi Lightweight Concrete

The reason that the six section combinations with both normal and lightweight concrete require varying initial strengths is due to the effective prestressing at the time of release. As discussed in the preceding discussion, the initial strengths required decrease with increasing amounts of lightweight concrete. From Figure 3.16, which gives the number of strands required for each section, it can be seen that the all normal weight concrete section and the section with the lightweight concrete girder and normal weight concrete deck require the same number of prestressing strands. However, the initial concrete strength required for the lightweight girder is approximately 7 percent less. Figure 3.17 shows that the initial prestress losses for the sections with the lightweight concrete girders are approximately 1.8 times that for the section with normal weight girders. This higher loss causes less effective compressive stress to be applied in the end region of the girder. Even though prestress losses are normally considered as a negative effect, higher prestress losses in this case results in a lower required initial concrete strength. This may also be viewed from a different perspective. If the losses for the lightweight concrete were not as high, then less strands would have been required and hence lower compressive stresses would have resulted anyway.

Further discussing prestress losses, it appears that as a result of the higher losses, minimal to no savings in strands will be realized. This is identical to results obtained for the 7500 psi concrete girders. The higher prestress losses in the lightweight concrete girders are mainly due to greater elastic shortening due to the lower modulus of elasticity that is typical of this material. For this reason, only a total of 8 strands can be eliminated in going from a section with all normal weight concrete to a section with all lightweight concrete. However, even though this appears minimal, in a very large structure these savings in strands may actually be significant. Closer examination of the initial prestress losses shows that the ratio of the average lightweight concrete girder losses to the average of the normal weight concrete girder losses is 1.68, whereas the ratio for the final losses is only 1.18. This difference is explained by examining the prestress losses that combine to make up the initial losses. The initial prestress loss is determined by adding the elastic shortening to one-half of the steel relaxation losses. For a lightweight concrete girder, the elastic shortening loss is the largest contributor to the prestress losses. Because of this, initial losses are going to be much larger than the final losses. Hence, as previously mentioned, application of lightweight concrete girders might yield greater effective use of the lower density if prestressing is applied once the concrete has achieved greater maturity. This would be possible in a post-tension application or possibly even in a two-stage stressing application. The difference is that in the post-tensioned girder the elastic losses occur prior to anchoring the tendon and thus are replaced by the much lower anchor set. In addition, the girder will have a higher f’ci at stressing, which results in a higher Ec and lower losses.

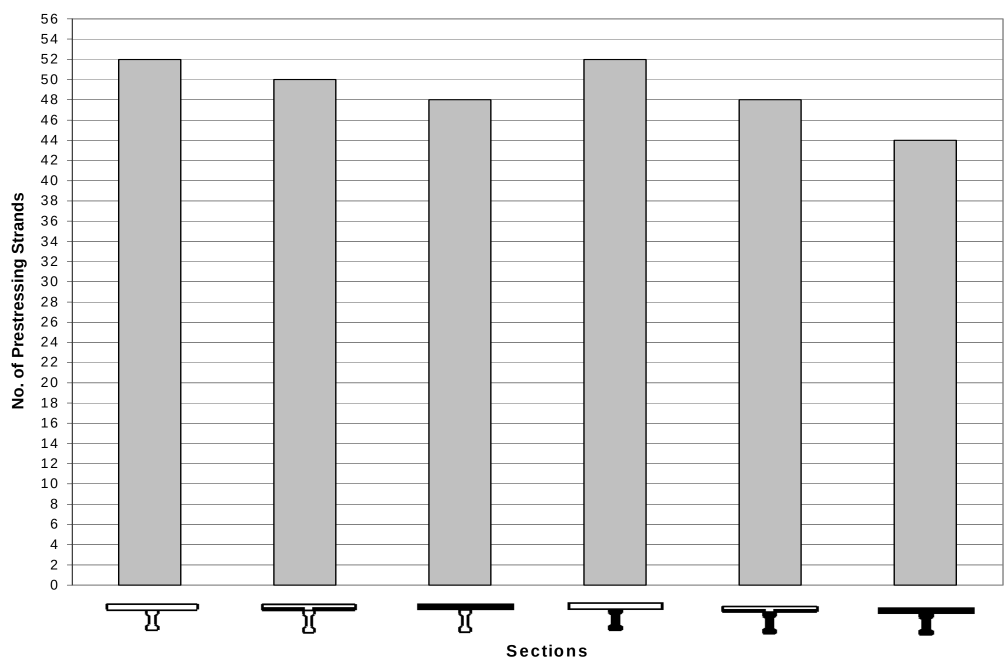


Figure 3.16 Prestressing Strand Requirements for 6000 psi Girders with Constant Span Length

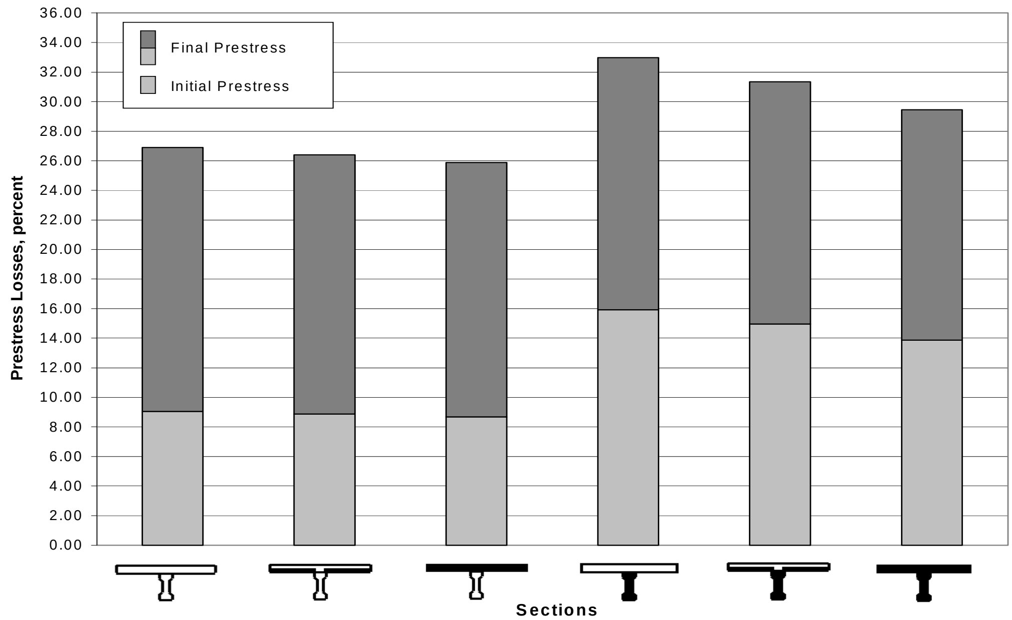


Figure 3.17 Prestress Losses for 6000 psi Girders with Constant Span Length

3.6.2.3 Flexure and Shear

Figure 3.18 shows the variation amongst the various sections for both the ultimate moment required and provided. From this plot, it is evident that a considerable amount of reserve capacity has been provided. In fact, the ratio of the provided moment to the required moment is approximately 1.40. This is because in these 6000 psi girders with a constant span length, design was clearly governed by the allowable stresses.

Shear stirrup spacing as determined by PSTRS14 is equal to 12 inches for all section combinations. However the lower tensile strength of the lightweight concrete must be incorporated into the shear calculations as indicated in AASHTO 9.20.2.5. The PSTRS14 program does not provide a means to incorporate these splitting tensile strengths. Hence, it is recommended that verification calculations be made externally to check shear requirements. Finally, there does not appear to be a great difference between the ultimate horizontal shear stresses for all section combinations. Table 3.13 indicates that the horizontal shear stresses for the normal and lightweight concrete sections vary from a high of 242 psi to a low of 202 psi.

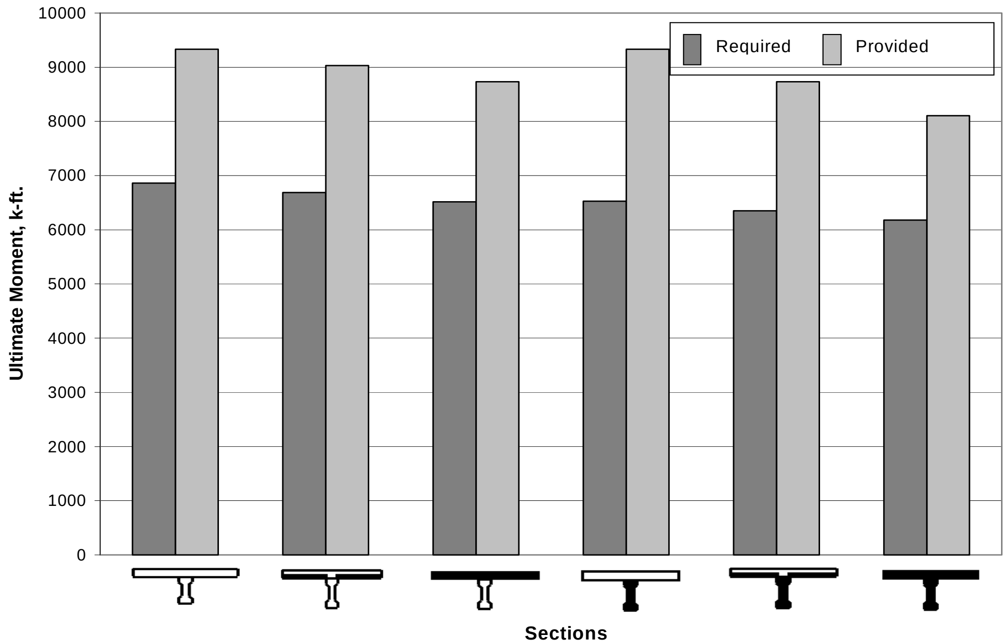


Figure 3.18 Ultimate Moment Required and Provided for 6000 psi Girders with Constant Span Length

3.6.2.4 Camber and Deflection

The camber for the various sections is given in Figure 3.19. The ratio of the average camber for the lighweight concrete, 5 inches, over the average camber of the normal weight concrete, 3.8 inches, indicates an approximate 32 percent increase for the lightweight concrete. A hand check of the camber calculated using multipliers could possibly yield differences from the camber estimated by PSTRS14 given the fact that so many variables influence camber. It would be prudent to monitor the camber of lightweight concrete members put into service to try and obtain a greater understanding of this property.

The variation of instantaneous elastic deflections due to dead load for the different sections is shown in Figure 3.20. The deflections for the lightweight concrete girders are on average about 43 percent larger than for the normal weight concrete girders. However, the average values range from approximately 2.1 inches for the normal weight concrete girders to 3 inches for the lightweight concrete girders. This difference in deflections between the normal weight concrete girders and the lightweight concrete girders can be attributed to the approximately 30 percent lower modulus of elasticity for the lightweight concrete. Also, to check that a net downward displacement is not going to exist under the self-weight of the girder and slab, the net deflection has been determined and is given in Figure 3.21. The net deflections are determined by taking 80 percent of the calculated slab dead load and subtracting it from the calculated camber. The difference should be greater than 1 inch. Examination of this figure, indicates that all sections are well above the established lower bound value of 1 inch.

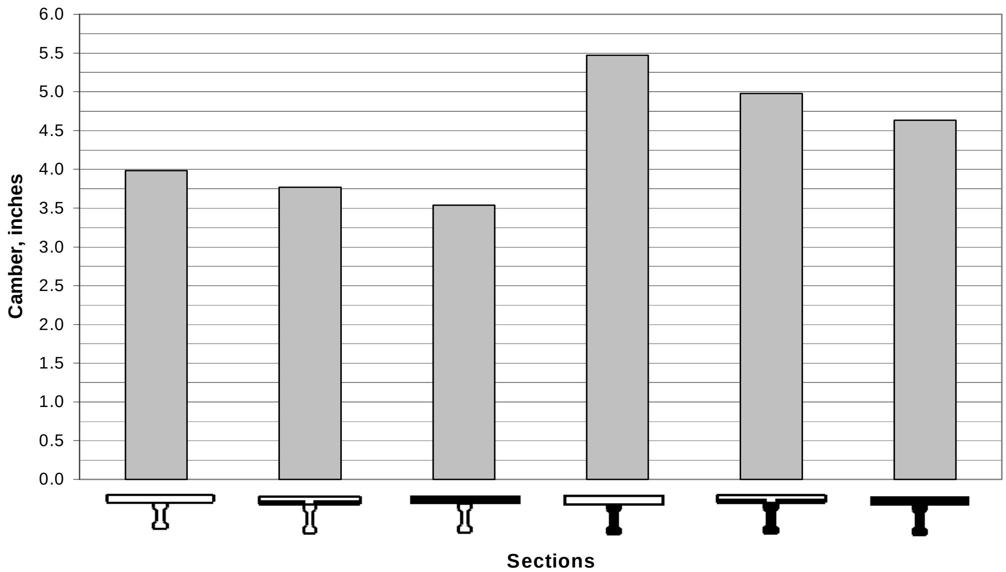


Figure 3.19 Camber for 6000 psi Girders with Constant Span Length

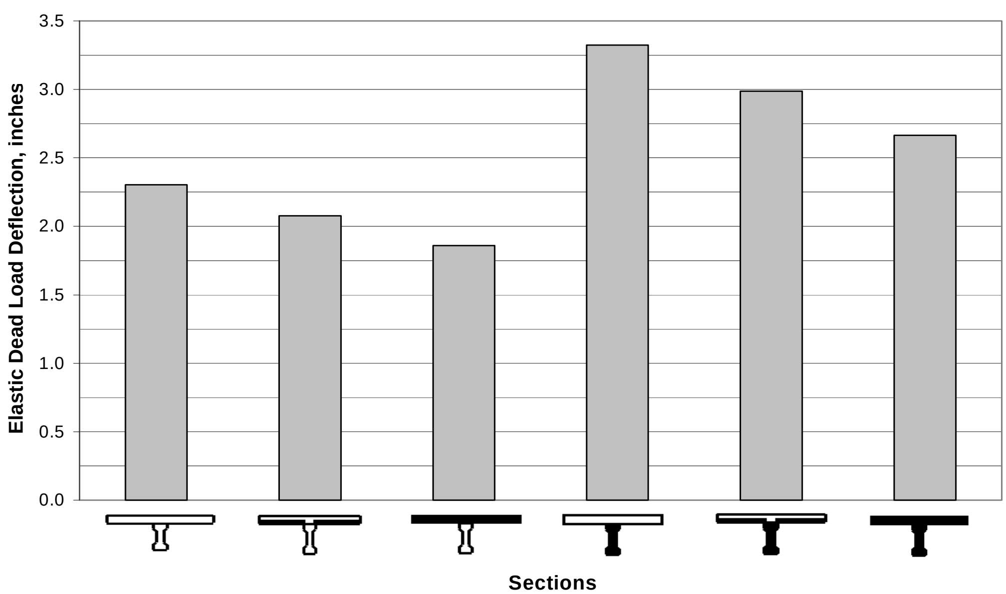


Figure 3.20 Elastic Deflections due to Dead Load for 6000 psi Girders with Constant Span Length

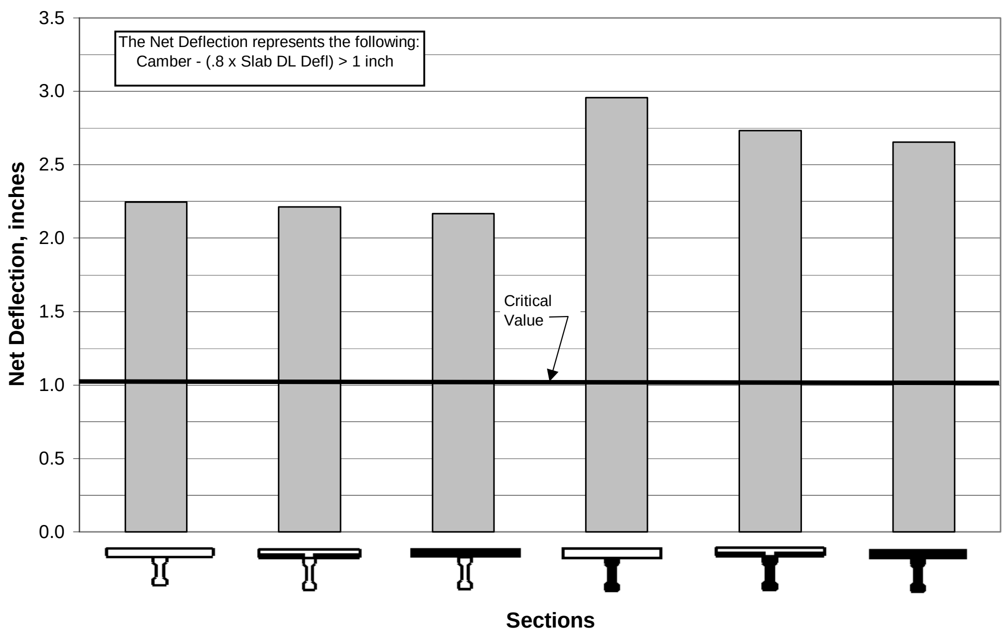


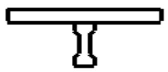
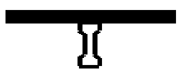
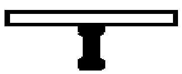
Figure 3.21 Net Deflections for 6000 psi Girders with Constant Span Length

3.6.3 Analysis Results for Constant Initial Strength

3.6.3.1 Section Properties

In this section of the report, girders with a 28 day concrete compressive strength of 6000 psi and an initial strength of 4000 psi are analyzed. The girders in this section differ from those previously discussed by the fact that the initial 4000 psi strength is maintained as a constant and girder spacing is 8.5 feet while the span length is varied. The different maximum span lengths can be found in Table 3.15.

Table 3.15 Section Properties for 6000 psi Girders with Constant Initial Strength

Section	Max. Span Length	Girder Spacing	f'c girder	f'ci girder	f'c slab	Girder Unit Weight	Slab Unit Weight	Ec girder	Ec slab	n Mod. Ratio	I girder	I' girder + deck
	(feet)	(feet)	(psi)	(psi)	(psi)	(pcf)	(pcf)	(ksi)	(ksi)		(in^4)	(in^4)
	90	8.5	6000	4000	5000	150	150	4696	4287	.913	260,403	687,810
	90	8.5	6000	4000	5000	150	134	4696	3406	.725	260,403	637,288
	95	8.5	6000	4000	5000	150	118	4696	2525	.538	260,403	574,364
	95	8.5	6000	4000	5000	118	150	3250	3250	1.00	260,403	708,041
	95	8.5	6000	4000	5000	118	134	3250	3250	1.00	260,403	708,041
	100	8.5	6000	4000	5000	118	118	3250	2525	.777	260,403	652,257

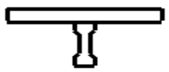
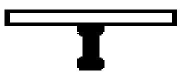
3.6.3.2 Prestressing Results

The prestressing strand properties used in the analyses of the 6000 psi girders with constant initial strength are given in Table 3.16. Pretensioning strand properties have remained the same as in the other analyses. Strand numbers and geometry are given in Table 3.17. The distance from the centerline of the girder to the hold-down point is the only other variable that differs for this set.

Table 3.16 Prestressing Variables Input into PSTRS14 for 6000 psi Girders

Prestressing Variables	
Variable	**Value**
No. of Strands	Varies, see Table 3.17
Strand Eccentricty (Center)	Varies, see Table 3.17
Strand Eccentricity (End)	Varies, see Table 3.17
Strand Size	½ inch
Strand Type	7-Wire Lo-Rlx
Strand Area	0.153 sq. inches
Strand Ultimate Strength	270 ksi
Es	28000 ksi
No. of Straight Web Strands	0
No. of Web Strands/Row	2
Relative Humidity	50 percent
Dist. CL to Hold Down	5.00 feet

Table 3.17 Prestressing Results for 6000 psi Girders with Constant Initial Strength

Sections	No. of Strands	Strand Eccentricity		Prestress Losses		Stress due to Tot. External Load		Stress @ End (Release)		Stress @ CL (Final)	
		End	CL	Release	Final	Top	Bott	Top	Bott	Top	Bott
		(in)	(in)	(percent)	(percent)	(psi)	(psi)	(psi)	(psi)	(psi)	(psi)
	30	11.95	21.15	7.08	19.93	2526	-2858	-64	2076	1702	-418
	30	11.95	21.15	7.08	20.21	2481	-2786	-64	2077	1660	-354
	32	12.38	21.00	7.27	20.75	2735	-2993	-112	2248	1878	-428
	32	12.38	21.00	11.28	24.30	2517	-2913	-107	2150	1699	-463
	30	11.95	21.15	10.60	23.21	2383	-2800	-62	1998	1593	-460
	34	12.75	20.87	11.58	25.28	2571	-2968	-153	2310	1724	-408

3.6.3.3 Flexure and Shear Results

The "Ultimate Moment Required" was an input into the PSTRS14 program. This variable consists of the moments due to dead loads acting on the girder, including the girder self-weight, as well as moment due to AASHTO HS20 live load. The program then determines the "Ultimate Moment Provided" based on geometric properties of the composite section, strand centroid, material properties of pretensioning steel and concrete. The program also determines 1.2 x Mcr and compares it to the "Ultimate Moment Required". As in the previous analyses of 6000 psi and 7500 psi girders, 1.2 x Mcr never governed. Table 3.18 contains the results for the flexure and shear calculations as determined by PSTRS14.

3.6.3.4 Camber and Deflection

Cambers and deflections are tabulated in Table 3.19. Recall that camber is a function of the eccentric prestress force. Because this is the case, prestress force will in turn be influenced by prestress losses. The prestress losses for lightweight girders are greater. Hence this would lead one to believe that the camber for these girders might be smaller. However, the elastic modulus for the lightweight concrete is also smaller, thereby increasing the upward deflection. Also, even though this is a comparative analysis, it will be difficult to get a true comparison of cambers and deflections because of the different span lengths. The lightweight concrete with the longer span lengths that are a result of the lower densities will most certainly have higher cambers and deflections.

Table 3.18 Flexure and Shear Analysis Results for 6000 psi Girder with Constant Initial Strength

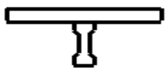
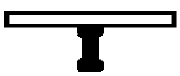
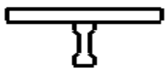
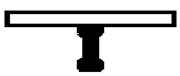
Sections	Shear Stirrup Spacing		Ultimate Horiz. Shear Stress	Ultimate Moment Required	Ultimate Moment Provided
	Near End	Near CL			
	(in)	(in)	(psi)	(k-ft)	(k-ft)
	10	12	219.2	5022	5764
	11	12	204.2	4906	5764
	12	12	189.2	5203	6113
	10	12	218.7	5212	6113
	10	12	213.5	5083	5764
	11	12	202.5	5351	6460

Table 3.19 Cambers and Deflections for 6000 psi Girders with Constant Initial Strength

Sections	Maximum Camber	Dead Load Deflections (Centerline)		
		Slab	Other	Total
	(ft)	(ft)	(ft)	(ft)
	.146	-.080	-.005	-.085
	.146	-.072	-.005	-.077
	.165	-.079	-.007	-.086
	.235	-.144	-.008	-.153
	.213	-.129	-.008	-.137
	.266	-.140	-.011	-.151

The analyses for the 6000 psi girders were based on predetermined variables and cross sectional properties. The results presented in this section are for the normal and lightweight concrete mixes in which a constant initial strength of 4000 psi was maintained. This resulted in girder span lengths less than the predetermined span length of 110 feet. The 110 feet span length can only be achieved if the 1-day strength is increased to a value greater than 4000 psi.

3.6.4 Discussion of Analysis

The analyses in this section were based on comparing 6000 psi girders made from both normal and lightweight concrete and with varying composite deck combinations. The initial concrete strength of 4000 psi was too low to satisfy design requirements for a span length of 110 feet. Because of this, in this section of the analysis the span length was reduced to a length that would allow for use of the mix design as developed (without any alteration of the concrete mix design to achieve a higher one-day strength). As discussed in a previous section, the strength at one day is an important factor for a prestressed concrete plant to maintain certain levels of production. The 4000 psi initial strength satisfies the 3500 psi one-day strength necessary for release of the strands, but was not sufficient to satisfy allowable stress criteria.

A discussion of the findings for this girder series will be presented in the following sections. However, because the span lengths vary for each section combination, it is difficult to make accurate comparisons between the six composite sections. Because of this, only a very brief discussion is warranted.

3.6.4.1 Section Properties

Figure 3.22 shows the maximum allowable span lengths for the various section combinations. These maximum span lengths are based on maintaining the 4000 psi initial strength and satisfying the allowable stress criteria as set forth in AASHTO. This figure shows that the lower density of the lightweight concrete allows girders made from this material to span a longer distance. The use of lower density lightweight concrete in the slab also allows increases in span length.

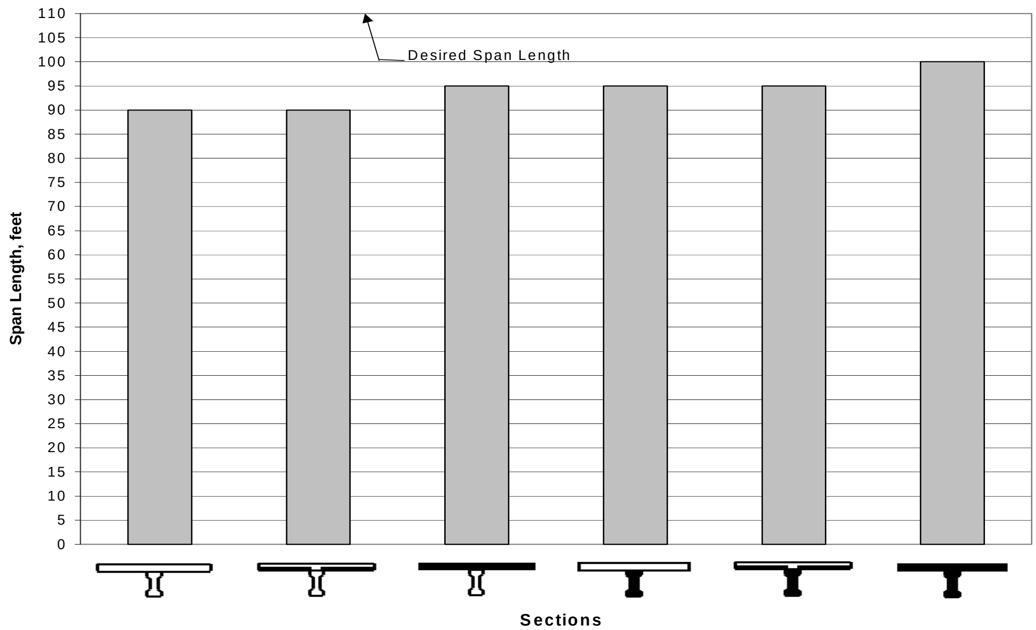


Figure 3.22 Maximum Span Lengths for 6000 psi Girders with Constant Initial Strength

3.6.4.2 Prestressing, Shear, Moment, Camber, and Deflection Properties

The prestressing results in Table 3.17 shows a maximum difference of four required prestressing strands between the all-normal weight concrete and the all-lightweight concrete sections. Because these two sections represent the extreme limits of the composite sections, this difference in strands is the most that will be realized in comparing any two sections. Also in Table 3.17, examination of the final stresses at the centerline for all-normal and all-lightweight concrete sections shows that the tensile and compressive stresses for each of the sections are almost identical. This is despite the lightweight girder being 10 feet longer than the normal weight girder. This is one of the clear advantages of using the lower density lightweight concrete.

In Table 3.18, it is interesting to note that the ultimate moment provided is about 15 to 20 percent larger than the ultimate moment required. In comparison, the ratio of the provided moment to the required moment for the 7500 psi sections and the 6000 psi sections with a constant span length was between 30 and 37 percent larger. The only explanation that can be offered for this difference is that longer spans with a larger number of prestressing strands provide proportionally larger moment capacity. As an example, a number of all-normal weight girder designs were performed with varying lengths. The resulting moment provided and moment required for each girder was then plotted against the number of strands required in Figure 3.22. From this figure, it can be seen that as the number of strands increase the difference between these moments gets larger. An unrealistic length of 60 feet (14 strands) for a Type IV girder was included to exaggerate the difference. Similarly, in Figure 3.23 the moments are plotted versus the span length and the same general trend is observed.

In Table 3.19, comparing the camber ratio of the all-lightweight section to the all-normal weight section shows a more than 80 percent increase. However, comparing the actual camber values of 3.2 inches for the all-lightweight concrete girder and 1.8 inches for the all-normal weight concrete girder reveals an increase of only 1.4 inches. Also, recall that the lightweight concrete girder is 10 feet longer than the normal weight concrete girder. Elastic dead load deflections also show an 80 percent increase with the deflection for the lightweight concrete girder equal to 1.8 inches and the deflection for the normal weight concrete girder equal to 1.0 inches.

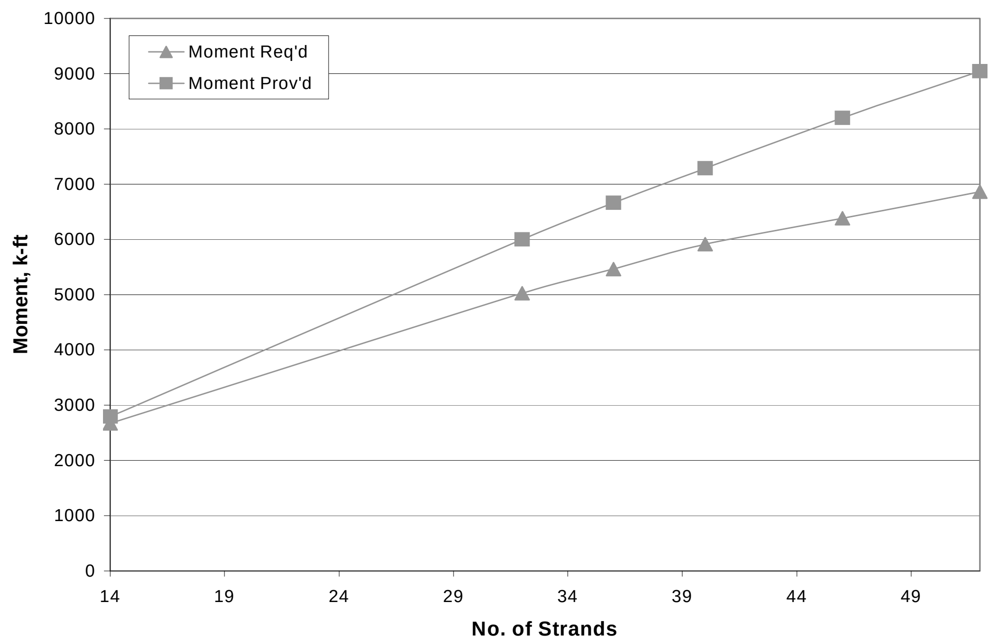


Figure 3.23 Variation of Moment Provided and Required to No. of Strands

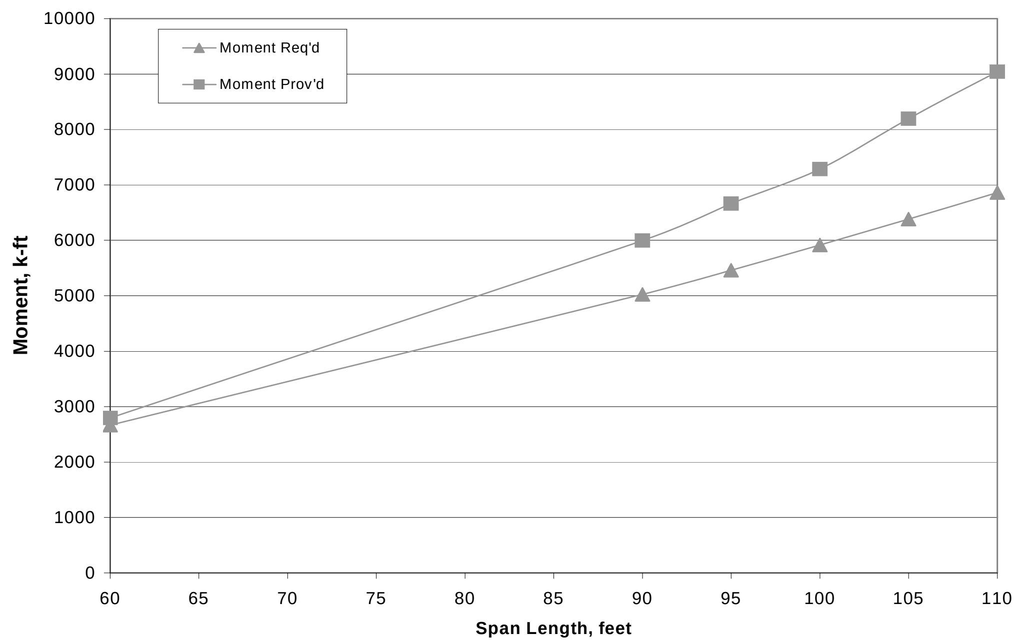


Figure 3.24 Variation of Moment Provided and Required to Span Length

CHAPTER 4

Economy and Implementation

4.1 INTRODUCTION

The engineering property that makes lightweight concrete a viable alternative to normal weight concrete is its lower density. The density of lightweight concrete is approximately 80 percent that of normal weight concrete. This lower density creates opportunities for cost savings in both the design and construction phases. In the design phase, the potential for savings includes lower pretensioning steel requirements in prestressed girders and lower overall dead load from the girders and the deck. The lower dead loads may allow larger beam spacing and smaller loads being transmitted to the substructure and the foundation. In construction, the lower density may result in cost savings due to easier handling and the potential for a reduction in shipping costs. Another advantage during construction is that the lower density may allow lifting of members that would otherwise be too heavy for the crane capacity. Hence, lightweight concrete offers more than cost savings, it offers opportunities to overcome constructability issues as well.

In this chapter, economy and implementation of lightweight bridge girders and lightweight precast concrete panels will be discussed. This discussion will include material availability, material cost, and plant production factors. Design benefits will be included followed by net economic changes. The chapter concludes with general design guidelines and a review of current TxDOT specifications and standards to determine if any changes are needed to incorporate lightweight concrete.

4.2 MATERIAL AVAILABILITY

The lightweight concrete mix designs were developed by Heffington [4] for this project. The concrete mixes consisted of many materials that are readily available and commonly used for concrete mixes, regardless of the type. These materials include a Type III cement, Type C fly ash, river sand, retarder, superplasticizer, and water. The only other material used in the mix design, which is available from only one producer in Texas, is the lightweight coarse aggregate. Three different lightweight coarse aggregates were used in the mix design development as follows:

- Clodine- an expanded clay manufactured by Texas Industries (TxI) in their plant south of Houston, Texas;
- Streetman- an expanded shale produced in the TxI plant south of Dallas, Texas;
- Western- an expanded clay obtained from a TxI subsidiary in Colorado.

Even though all three of these lightweight aggregates were used in batch trial testing, Clodine was eventually chosen as the aggregate most suitable for the lightweight concrete developed in this project.

4.3 MATERIAL COST

Figure 4.1 shows estimates of the premium cost of lightweight concrete obtained from industry sources (mostly precast concrete product suppliers) in Texas. The premium cost is the differential material cost between normal weight concrete and lightweight concrete. From this figure, the premium cost estimates range from $6/cy to $30/cy, with an average of about $18.50/cy. In addition, information was obtained from a contractor that used lightweight concrete in a bridge deck replacement in the Houston, Texas area. The premium cost of lightweight concrete for the project was $55/cy. However, this unit price includes the price of delivery to the job site.

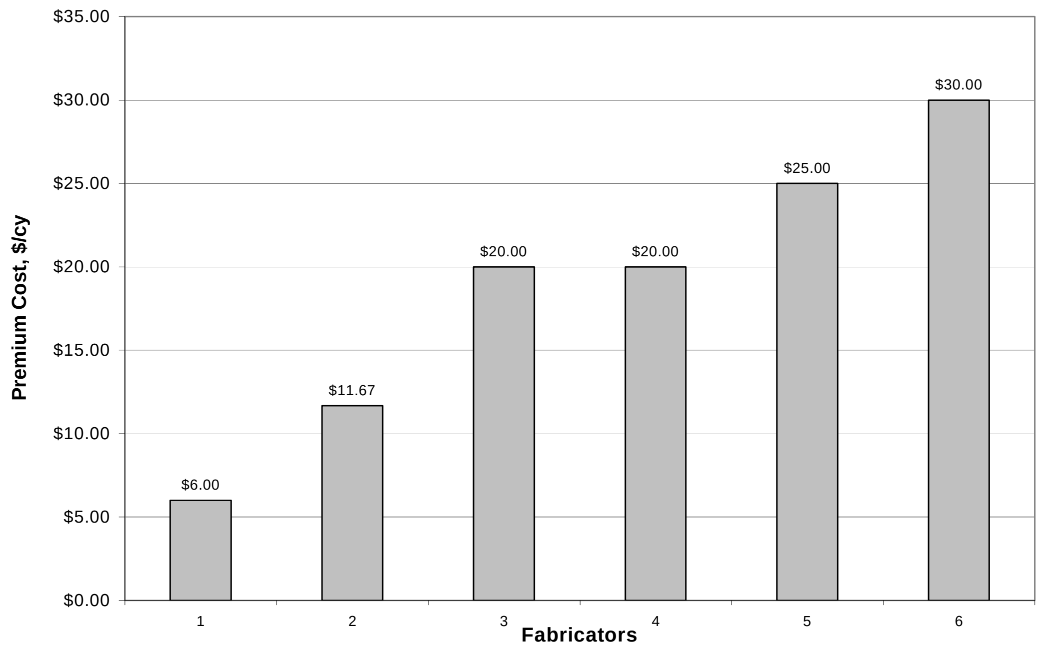


Figure 4.1 Premium Cost of Lightweight Concrete

4.4 PLANT PRODUCTION FACTORS

Because most producers of precast pretensioned bridge girders in Texas do not currently use lightweight concrete, there are operational considerations that need to be made before implementation. These include, but are not limited to, the following items:

- plant space required to maintain lightweight aggregate stockpiles
- moisture control of lightweight aggregate stockpiles (Figure 4.2)
- QC/QA requirements
- plant bin/hopper space
- inspection for bin inventory management
- storage tanks for fly ash and cement

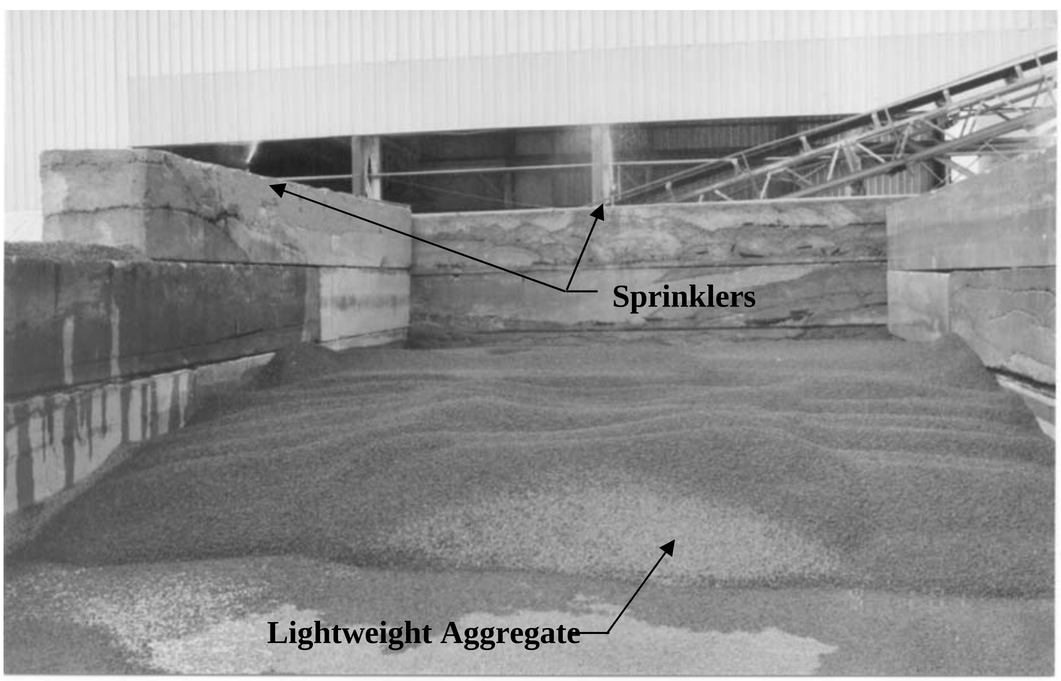


Figure 4.2 Lightweight Aggregate Stockpile with Moisture Control

Other items that might affect precast producers who are not currently using lightweight concrete include concerns about availability of lightweight coarse aggregate, reliability of the lightweight aggregate, and concerns over limited production. Limited production is a concern because implementation of girders and panels made from lightweight concrete will more than likely require some investment on the part of the producer.

Even though the are many logistical factors to consider, it must be pointed out that there are precast producers of building products, such as double-T beams, that currently use lightweight concrete on a daily basis in Texas. These producers have found that some of the economies, such as reduced shipping costs, make lightweight concrete a viable alternative.

4.5 DESIGN BENEFITS

The design benefit of lightweight concrete is mostly due to the lower density of this material. The lower density of panels and girders translates into lower dead load on the substructure. The lower dead load on the substructure means potentially smaller substructure members and lower foundation bearing loads. It would also appear that the lower dead load would make it possible to have fewer beams per span. However, as shown in Table 4.1 both the normal concrete and lightweight concrete girders have about the same design capacities. This table was compiled from results of an analysis of bridge sections with 58 feet and 70 feet total widths. This section is shown in Figure 4.3. The analysis was based on determining the largest spacing that could be achieved with an all-normal weight section. The next higher spacing (lower number of beams) was then used to check what designs could be achieved with lightweight concrete. This included varying the amount of lightweight concrete in the deck, as well as using lightweight concrete girders. From the results, it appears that the greatest impact for achieving the larger spacing is due to increasing the amount of lightweight in the deck. The reason that the lightweight girder has the same design capacity as the normal weight girder is due to the higher prestress losses and the lower allowable stresses. In this analysis, the more conservative allowable stresses were used for the lightweight concrete girder design. The initial allowable stress in tension was limited to 6.3 times the square root of the 1-day compressive strength (f'ci). The final allowable tensile stress was limited to 5.0 times the square root of the 28-day compressive strength (f'c). These are both about 17 percent lower than the allowable tensile stresses for normal weight concrete.

No. Travel Lanes (12 ft.)	Total Width (ft.)	Beam Spacing ~ X				
		6 Beams (ft.)	7 Beams (ft.)	8 Beams (ft.)	9 Beams (ft.)	10 Beams (ft.)
3	58	10.40	8.67	7.43	6.50	5.78
4	70	12.80	10.67	9.14	8.00	7.11

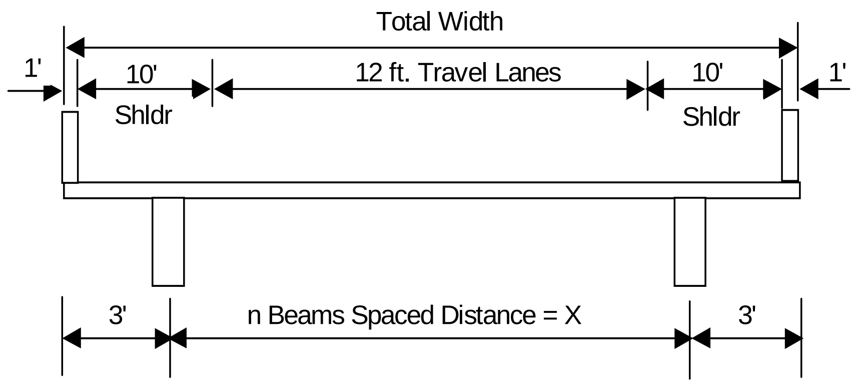


Figure 4.3 Prestressed Concrete Girder Section

Table 4.1 Design of Prestressed Girders with Wide Spacings

Total Width	Girder Matl.	Deck Matl.	No. of Girders	Girder Spacing	Acceptable Design	Unacceptable Design
58 ft.	NW	NW	7	8.67 ft.		✔
	NW	NW/LW	7	8.67 ft.	✔	
	NW	LW/LW	7	8.67 ft.	✔	
	LW	NW	7	8.67 ft.		✔
	LW	NW/LW	7	8.67 ft.	✔	
	LW	LW/LW	7	8.67 ft.	✔	
70 ft.	NW	NW	8	9.14 ft.		✔
	NW	NW/LW	8	9.14 ft.		✔
	NW	LW/LW	8	9.14 ft.	✔	
	LW	NW	8	9.14 ft.		✔
	LW	NW/LW	8	9.14 ft.		✔
	LW	LW/LW	8	9.14 ft.	✔	

Although this project did not investigate long-term effects such as durability, permeability, resistance to ASR and sulfate attack, a literature review indicated that lightweight concrete exhibits behavior equal to or better than normal weight concrete for all of these properties. From the research done by Vaysburd, tests showed that the lightweight concrete had lower permeability than normal weight concrete [12]. From a report done by Holm and Bremner, they report that from investigations of normal density concrete and low density concrete exposed to the same testing criteria that the low density concrete had equal or lower permeability [13]. In the same Holm and Bremner report, it is stated that “concrete made from either natural LDA or manufactured LDA appears not to be adversely affected by any long-term interaction between silica-rich aggregates and the alkalies in the cement” [13]. Freeze-thaw durability, marine durability, atmospheric durability, and corrosion protection to reinforcement are either equal or superior to that provided by normal weight concrete according to Ben C. Gerwick [14].

4.6 NET ECONOMIC CHANGES

Cost information was obtained from a Texas precast producer of concrete bridge girders. This cost information was established based on a hypothetical bridge project with a shipping distance of about 40 miles. The project consisted of a six span bridge with ten AASHTO Type IV girders per span. The span length of these girders was 110 feet. These girders are similar to those used in the analyses.

Prices were established for both normal weight concrete and lightweight concrete girders. Each of these girders was estimated with concrete strengths of 6000 psi and 7500 psi. The estimated prices are given in Table 4.1 and show that the lightweight concrete girders are about 10 to 15 percent higher than the girders made from normal weight concrete. For this example, reduction of shipping cost was not enough to completely offset the higher material cost of the lightweight concrete.

According to the prestressed girder manufacturer, the reduced load of lightweight girders would reduce shipping cost by approximately 15 percent. This may offset the higher material cost of lightweight concrete if it is shipped long distances. But as was shown for shipping distances of about 40 miles, it is not enough to offset the higher material cost. However, a greater reduction in shipping cost, 35 to 40 percent, would be possible if two girders could be placed on the same truck.

Table 4.2 Unit Price of AASHTO Type IV Bridge Girders

Concrete Type	Concrete Strength (psi)	Girder Unit Price ($/LF)
Normal	6000	51.35
Lightweight	6000	56.50
Normal	7500	51.25
Lightweight	7500	59.25

Placing two girders on the same truck is one of the reasons that lightweight concrete can be competitive for precasters that make double-T beams. Figure 4.4 represents the shipping weights of three different types of precast concrete girders. The concrete strength of these girders was assumed to be 7500 psi. Hence the unit weight is 122 pcf. From this figure, it can be shown that the only viable girder type that can practically take advantage of reduced shipping costs is the Type A girder. This is because each girder must weigh 25000 lbs. or less so that the 50,000 lbs. shipping limit is not exceeded. Figure 4.5 shows the same results for 6000 psi girders.

Because precast concrete panels are laid flat and stacked when they are shipped, there is greater potential for placing more members on a transport vehicle. Hence, reduced shipping costs are quite possible for these members. Assuming, again that the same 50,000 lbs. shipping limit is applicable to the panels, Figure 4.6 shows the number of lightweight concrete panels that can be shipped in comparison to the normal weight concrete panels. However, this comparison is based strictly on shipping weight. The actual number of panels that can be shipped due to height and placement constraints on the transport vehicle may cause this number to vary. This is simply an example to show the reduced shipping cost benefits from using the lower density concrete.

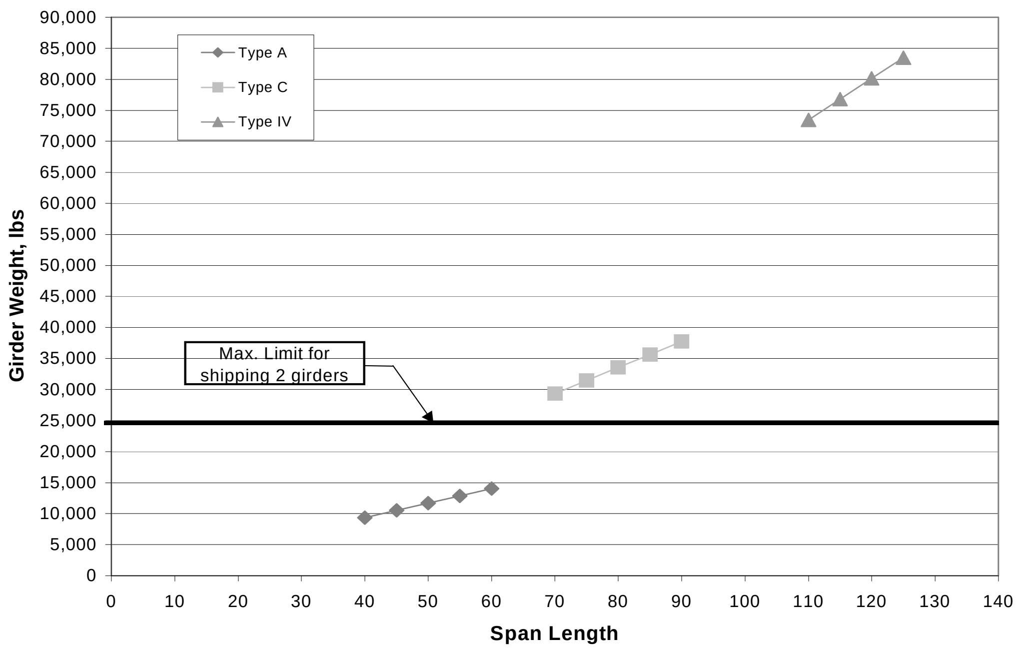


Figure 4.4 7500 psi Precast Concrete Girder Shipping Weights

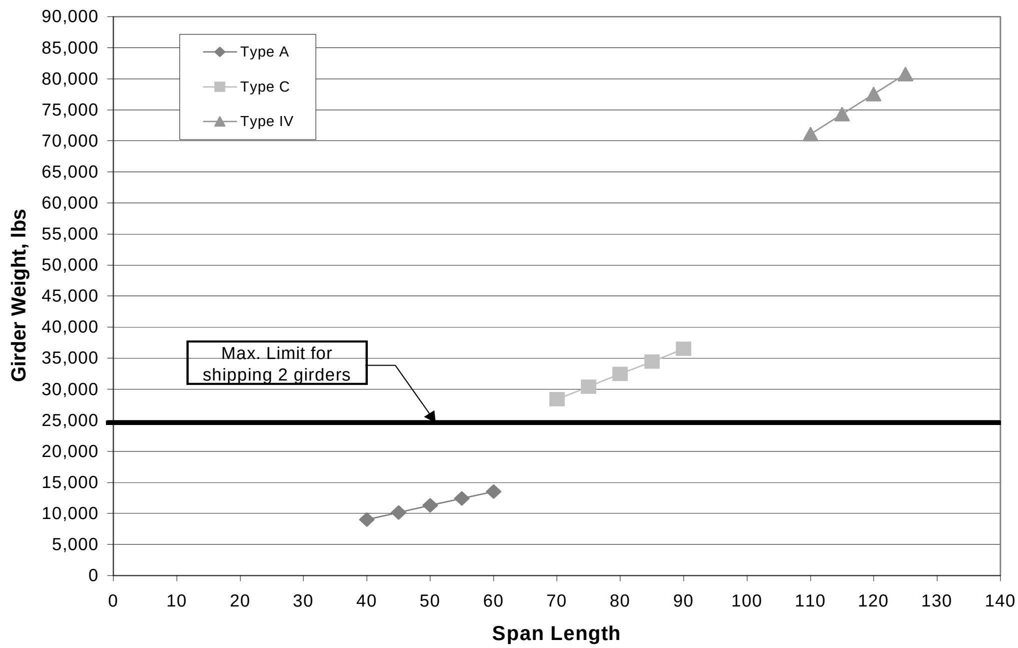


Figure 4.5 6000 psi Precast Concrete Girder Shipping Weights

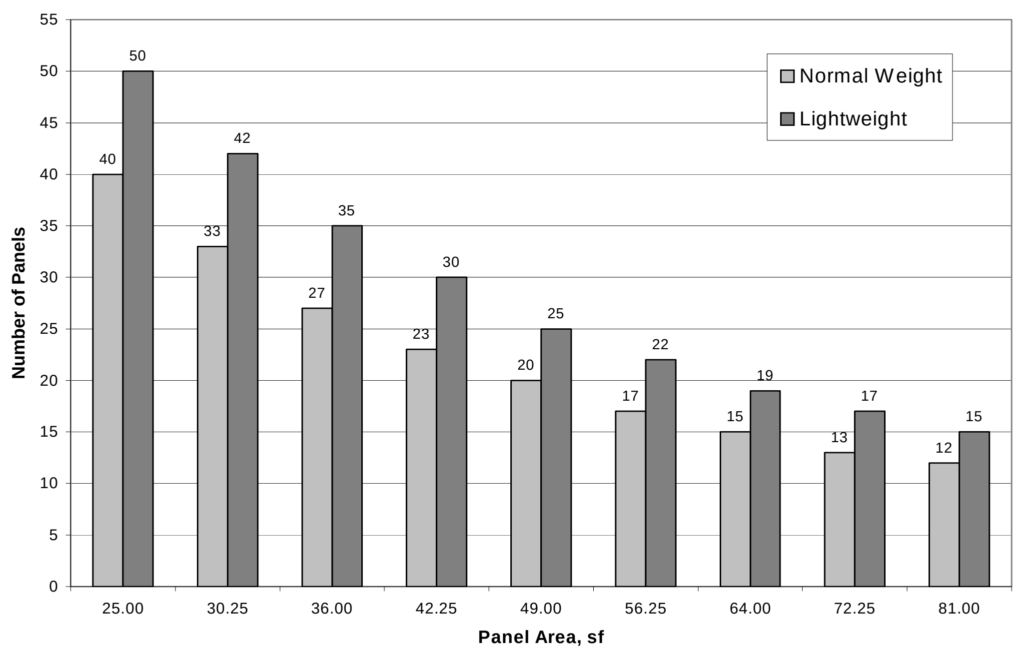


Figure 4.6 Comparison of Precast Concrete Panels for Shipping

The values in the figure were established by dividing the 50,000 lbs. shipping weight by the density of the concrete and the thickness of the panels (standard 4 inches). This gave the total surface area of panel that could be transported. This total surface area was then divided by taking panels that ranged in size from 5 feet square to 9 feet square with 6-inch increments in between. This ratio then gave the total number of panels shown in Figure 4.6. The larger difference is with the smaller panels, with the number gradually decreasing as the panel size increases.

Another potential net economic change is due to reduction of crane capacity. This however is difficult to quantify because it is dependent on each contractor's crane inventory or future crane rental. However, examining a girder similar to those used in the analyses herein, a total weight savings of nearly 10 tons is possible by going from a normal weight concrete (150 pcf) to a lightweight concrete (118 pcf). This reduction in dead load could also increase constructability of projects that are limited by crane lifting capacities.

4.7 DESIGN GUIDELINES

Because PSTRS14 is the primary prestressed concrete beam design program used by TxDOT, the design procedure used in these analyses will be given. Also, suggestions will be made that would make the program more versatile and less cumbersome for the design of lightweight concrete members will be given. In the present form of PSTRS14, design of prestressed lightweight girders requires verification of the results by hand. This is especially true for shear because the program does not allow input of the splitting tensile strength required for shear design of a lightweight concrete girder.

4.7.1 PSTRS14 Design Procedure

- Because the prestressed lightweight girder designs were based on specific mix designs, tested concrete compressive strengths and moduli of elasticity values should be used in the input. This will require that the analyze option in PSTRS14 be utilized since strengths have been predetermined;
- In the MAT1 card, input the following:
  - → unit weight of the beam
    [118 pcf (6000 psi); 122 pcf (7500 psi)]
  - → modulus of elasticity of the beam
    [3250 ksi (6000 psi); 3390 ksi (7500 psi)]
  - → unit weight of the slab, pcf
  - → modulus of elasticity of the slab, ksi
  - → final concrete strength of slab, psi
- Input prestressing strand using STPR and LOCR cards
  (number of prestressing strands will be an iterative process)
- In the ANLY card, input the following:
  - → beam top fiber stress at centerline, due to total external load, psi
  - → beam bottom fiber stress at centerline, due to total external load, psi
  - → ultimate moment required, k-ft
  - → specified beam concrete release strength, f'ci
    [4000 psi (6000 psi); 5500 psi (7500 psi)]
  - → specified beam concrete final strength, f'c

[6000 psi; 7500 psi]

- In SPEC card, input the following:
  - → 1 [column 10]
  - → initial prestress losses

    [these must be determined externally and input; losses will vary depending on the number of strands, hence they must be recalculated each time the number of strands is altered]
  - → final prestress losses

    [these must be determined externally and input; losses will varying depending on the number of strands, hence they must be recalculated each time the number of strands is altered]
  - → multiplier to determine initial tensile stress [6.3, suggested]
  - → multiplier to determine final bottom tensile stress [5.0, suggested]
- In BEAM card, input the following:
  - → span designation
  - → beam designation
  - → beam type
  - → span length, ft
  - → beam spacing, ft
  - → slab thickness, in.
  - → uniform dead load on composite section, due to overlay, klf
  - → uniform dead load on composite section, excluding overlay, klf

- vary the number of strands until "beam satisfies all design requirements" (prestress losses must be determined each time strand numbers change)
- make sure that modular ratio does not exceed unity
- verify shear stirrup spacing using split tensile strength and AASHTO 9.20.2.5
- check cambers and deflections

4.7.2 PSTRS14 Program Improvements

It appears that the PSTRS14 program could be improved to make design of lightweight girders less cumbersome. This would include logic that would limit the modular ratio to unity, allow input of the initial elastic modulus for prestress loss calculations, and allow input of split tensile strength for shear calculations. Other improvements may be required as a designer becomes more familiar with lightweight concrete. This may include improving camber estimation if predicted values are very different from actual field conditions.

4.8 SPECIFICATION/STANDARD REVISION

Before the Precast Concrete Panel Standard (PCP) is released for use with lightweight concrete, verification that the number of prestressing strands specified can adequately carry the load of the plastic concrete deck needs to be verified. This is necessary because of the lower tensile strength of lightweight concrete. The standard also needs to specify the class of concrete that is going to be used for lightweight concrete.

In addition, a Special Specification that addresses lightweight concrete needs to be prepared. This Specification should address the high slump of the lightweight concrete, the mix design proportions, and the mix design components. The coarse aggregate as well as the moisture control of the aggregates needs to also be addressed.

CHAPTER 5
Summary and Conclusions

5.1 SUMMARY

Lightweight concrete has a density that is approximately 20 percent less than normal weight concrete. This makes lightweight concrete an attractive candidate for use in bridges. The purpose of this report was to examine the advantages and disadvantages of using lightweight concrete in bridge girders and in precast concrete panels. Several analyses of a typical TxDOT bridge span were performed to compare the behavior of lightweight concrete girders and/or deck systems with that of normal weight concrete components. The results obtained from these analyses provided information useful for evaluation of the advantages and disadvantages of using lightweight concrete.

5.1.1 Panel Transfer Length

Laboratory testing of girders on this project by Kolozs [5] and Thatcher [6] revealed that the transfer length of girders made from lightweight concrete was longer than that of similar girders made from normal weight concrete. This raised the question of whether or not the 3/8-inch pretensioning strands in a lightweight concrete panel would have sufficient transfer length. To determine this, a test of three normal weight concrete and three lightweight concrete panels was performed.

The panels for testing were cast at a local precast plant along with panels for an upcoming bridge project. The lightweight concrete panels were finished and cured similarly to the normal weight concrete panels with no problems reported by the workers. After curing of the panels for approximately 18 hours, they were instrumented with points for strain measurements. Measurements were taken before release of the strands, after release, and 85 days later. The data revealed that the actual transfer length of the 3/8-inch pretensioning strand in the lightweight concrete panel is equivalent to approximately 45 strand diameters (d_b). This is in comparison to the $39d_b$ measured for 3/8-inch strands in normal weight concrete panels and the $50d_b$ specified by AASHTO 9.20.2.4. Hence, it was determined that the current AASHTO design rules and procedures for transfer length are conservative and can be used for both types of panels.

5.1.2 Beam Analysis

The comparative analyses of typical bridge span using AASHTO Type IV bridge girders and decks made from various combinations of normal weight and lightweight concrete were performed using the TxDOT prestressed girder design program, PSTRS14. Girders with 28-day compressive strengths (f'c) of 6000 psi and 7500 psi were analyzed for a standard TxDOT bridge section with an overall width of 40 feet and a span length of 110 feet. Three different composite deck combinations were used in the analyses of each one of these girders. The deck combinations consisted of an 8 inch all-normal weight concrete deck; a 4 inch normal weight concrete deck over a 4 inch lightweight concrete panel; and an 8 inch all-lightweight concrete deck.

From the analyses of the 7500 psi girders, it was determined that initial prestress losses were about 68 percent higher for the lightweight concrete girder as compared to the normal weight concrete girder. The final losses were approximately 21 percent higher. The higher losses were mainly due to the 35 percent lower modulus of elasticity typical of lightweight concrete. It was determined that the most sensitive variable in the prestress losses of the lightweight girder is elastic shortening, which is inversely proportional to the initial elastic modulus. Because of the higher losses, minimal to no prestressing strand savings can be realized between the normal weight concrete girders and the lightweight concrete girders. Flexure results for both normal weight concrete and lightweight concrete girders indicated that the provided moment was about 30 percent larger than the required moment. This indicates that the design of the girders was not controlled by flexure, but by allowable stresses. Shear results determined by PSTRS14 do not incorporate splitting tensile stress. Hence, an external design calculation must be performed to account for the lower tensile capacity of the lightweight concrete. The lower modulus of elasticity that affected the elastic shortening component of the prestress losses of the lightweight concrete also caused the cambers and deflections of the lightweight concrete to be larger. The average instantaneous dead load deflection determined for the lightweight girders is 2.9 inches, whereas the deflection of the normal weight girders averages 1.9 inches. This represents a 50 percent increase in deflections. The averages of the cambers for the normal weight and lightweight concrete girders are 3.4 inches and 4.8 inches, respectively. This represents a 40 percent increase in camber.

The analyses for the 6000 psi girders were not as straightforward as the analyses of the 7500 psi girders. Due to low initial strength (f'ci= 4000 psi) the 6000 psi girders did not satisfy allowable stresses for the predetermined span length and beam spacing. Because of this, an alternative analysis approach using the same standard sections was taken. First, the 110 feet span length was held constant and the initial strength (f'ci) was varied until allowable stresses were satisfied for each of the section. Second, the initial strength (f'ci) was held constant and the maximum span length was varied, again until allowable stresses were satisfied.

The first approach indicated that initial strengths for the 6000 psi concrete would need to be increased from 4000 psi to 5700 psi for the all-normal weight concrete section and 4825 psi for the all-lightweight section. All other required initial strengths for the other sections were determined to be within the range of strength of the all-normal and the all-lightweight concrete sections.

Using the first approach with a constant span length, nearly the same results for the prestress losses in the 7500 psi girders were obtained for the 6000 psi girders. The ratio of initial losses (lightweight to normal weight) was determined to be 1.68 and the ratio of the final losses was 1.18. Flexure results showed an approximate 40 percent reserve capacity in the girders, again reinforcing that the design was governed by allowable stresses. Average camber values determined for the lightweight concrete girders and the normal weight concrete girders were 5 inches and 3.8 inches, respectively. This is an approximate 30 percent increase in the camber of the lightweight girder. The instantaneous elastic dead load deflections were determined to be approximately 40 percent larger. The average lightweight girder deflection is 3 inches and the average normal girder deflection is 2.1 inches. Both the higher values of camber and deflection for the lightweight girder are due to the 30 percent lower modulus of elasticity.

In the second analysis approach with the initial strength held constant, it was determined that the maximum span length that could be achieved by the all-normal weight concrete section was limited to 90 feet. The maximum span length for the all-lightweight concrete section was determined to be 100 feet. The flexure results indicated that the difference between moment provided and moment required was only 15 to 20 percent for these girders. Even so, allowable stresses still controlled. The difference in the ultimate moment reserve capacity determined for these girders and the 30 to 40 percent determined for the longer 110 feet girders is most likely due to the difference in prestressing strands and span lengths.

5.1.3 Economic Analysis

Several Texas precast concrete product manufacturers supplied premium costs for lightweight concrete. A premium cost is the differential cost between lightweight concrete and normal weight concrete. From the information gathered, the range of these costs was from $6/cy to $30/cy, with the average of all cost equal to $18.50/cy. Unit costs for lightweight concrete girders, both 6000 psi and 7500 psi, and normal weight concrete girders were obtained from a Texas bridge girder manufacturer. The difference in unit costs for a lightweight concrete girder in comparison to a normal weight concrete girder was approximately 10 to 15 percent higher for girder made from lightweight concrete. The unit cost of the lightweight girders was $56.50 (f'c=6000 psi) and $59.25 (f'c=7500 psi). In comparison, the cost of the normal weight girders were $51.35 (f'c=6000 psi) and $51.25 (f'c=7500 psi).

Initially it was believed that this cost differential between the lightweight concrete girders and the normal weight concrete girders could be offset by reduced shipping costs. Over short shipping distances, the material cost of lightweight concrete will usually outweigh its reduced shipping cost. The greatest reduction in shipping costs would be achieved by placing two girders on the same transport vehicle. However, it was determined that this was only possible with the smaller AASHTO Type A girders.

5.2 CONCLUSIONS

The major findings that have resulted from the comparative analyses of lightweight concrete girders and panels with normal weight concrete girders and panels can be summarized as follows:

1. Higher prestress losses and lower allowable stresses reduced the efficiency of the lightweight concrete girders. Elastic shortening was determined to be the most sensitive prestress loss variable in lightweight concrete;
2. Lightweight girders made from the 7500 psi concrete mix design developed in this project had no problem satisfying allowable stress criteria and achieving the geometry of the standard TxDOT bridge section (110 feet span and 8.5 feet beam spacing) used in the analysis;
3. Lightweight concrete girders made from the 6000 psi concrete mix design could not satisfy allowable stress criteria due to the low initial strength. Hence, the maximum span length that could be achieved would be less than 110 feet;
4. Lightweight concrete panels, contingent upon verification of tensile capacity, could be implemented and could potentially produce savings in shipping and handling costs as well as reduction of dead load transmitted to the girders and substructure;
5. Premium cost of lightweight concrete ranged from \$6/cy to \$30/cy, with the average of all premium costs equal to \$18.50/cy;
6. Comparisons of normal weight concrete and lightweight concrete bridge girders shipped approximately 40 miles revealed 10 to 15 percent higher unit cost for the lightweight girders.

7. A design procedure for lightweight concrete has been recommended. The PSTRS14 program would require revisions to make it fully functional for the design of lightweight concrete beams.

5.3 RECOMMENDATIONS

From the results obtained in this study, the tradeoffs between the lower density of the lightweight concrete and the undesirable design factors such as higher prestress loss, lower tensile strength, and higher deflections and cambers must be carefully considered on a project specific basis. These factors appear to be more critical for the girders than for the precast concrete panels made from lightweight concrete.

If it is determined that the tensile capacity of the panel can safely carry the load of the plastic concrete, it would be prudent to begin using these panels in an actual field test project. Field testing is also highly recommended for the girders. Before implementation, it would be advisable to use and monitor the performance of the girders and panels in a limited number of actual field projects. The other alternative to consider is that the Virginia Department of Transportation (VDOT) is in the process of building a high performance lightweight concrete bridge that is expected to be completed at the end of year 2002. In their experimental investigation, being conducted by Virginia Tech, one of the objectives is to evaluate prestress losses associated with the high performance lightweight concrete. Because prestress losses are critical to the efficiency and the design of the lightweight girders, it is recommended that the results from the Virginia Tech study be compared to this study. Also, the knowledge and the experience gained from construction of this high performance lightweight concrete bridge should be a benefit to both designers and constructors for future lightweight concrete projects in Texas.

5.3.1 Future Study

Any future study of high performance lightweight concrete for use in bridges needs to examine the long-term effects of this material. This should at a minimum include comprehensive investigations of creep, shrinkage, and durability. If the present research at Virginia Tech determines inconclusive results of the prestress losses, this should also be considered. Because elastic shortening was determined to be the critical prestress loss variable in lightweight concrete, examination of post-tensioned applications could also be considered for future study as well.

5.4 IMPLEMENTATION

5.4.1 Recommendations

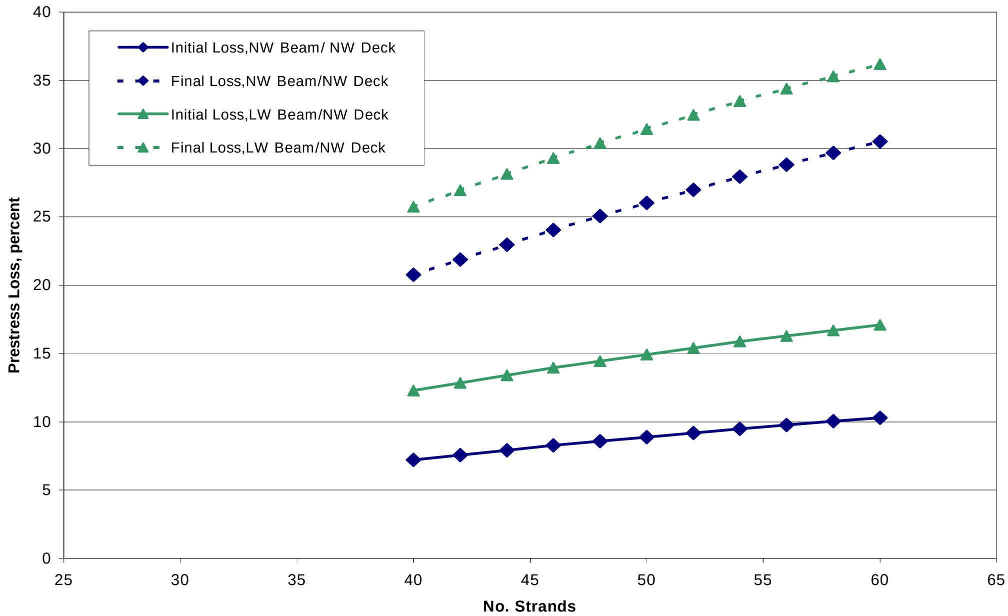
The implementation of lightweight concrete girders should begin by either identifying a limited number of candidate projects for field monitoring or by monitoring results from the high performance lightweight concrete bridge being constructed by the Virginia Department of Transportation (VDOT). Before implementation of the lightweight concrete panels, the tensile capacity of the current design for carrying the load of the plastic concrete slab needs to be determined. Results from the field-testing will benefit designers in determining whether or not high performance lightweight concrete is a viable alternative for bridge projects in Texas.

Table A summarizes some the results from the PSTRS14 analyses for the 7500 psi concrete beams. From this table, it is interesting to note that 50 strands are required for the section consisting of a normal weight beam with an all normal weight deck, as well as the section consisting of a lightweight beam with the same deck. Intuitively, one would think that the lightweight beam with the lighter dead load would result in a savings in the number of strands, but as will be shown in the figures that follow, higher prestress losses in the lightweight beam counteract this savings in dead load.

Table A: Summary of Prestressing Requirements for 7500 psi Beams

[redacted]

To gain a better understanding of the importance of the prestress losses for the lightweight beams, Figure A was developed. This figure depicts the variation of initial and final prestress losses as the number of prestressing strands for the normal and lightweight sections described above are varied between 40 and 60 strands.



<u>**Figure A: Comparison of Prestress Losses**</u>

From this figure, it is evident that the prestress losses are considerably higher for the lightweight beam. On average, the initial losses are approximately 68% higher for the lightweight as compared to the normal weight, while the final losses are approximately 21% higher. These higher losses in the lightweight beam can be attributed mostly to this materials lower modulus of elasticity.

Because of the higher prestress losses experienced by the lightweight concrete beam, the effective prestress force as determined by Equation 5.1 for this beam will also be lower.

$$.75 \times f_{pu} \times A_{ps} \times N \times \text{Total Losses} \qquad \text{(Equation 5.1)}$$

A comparison of effective prestress force between the normal and lightweight beams is shown in Figure B.

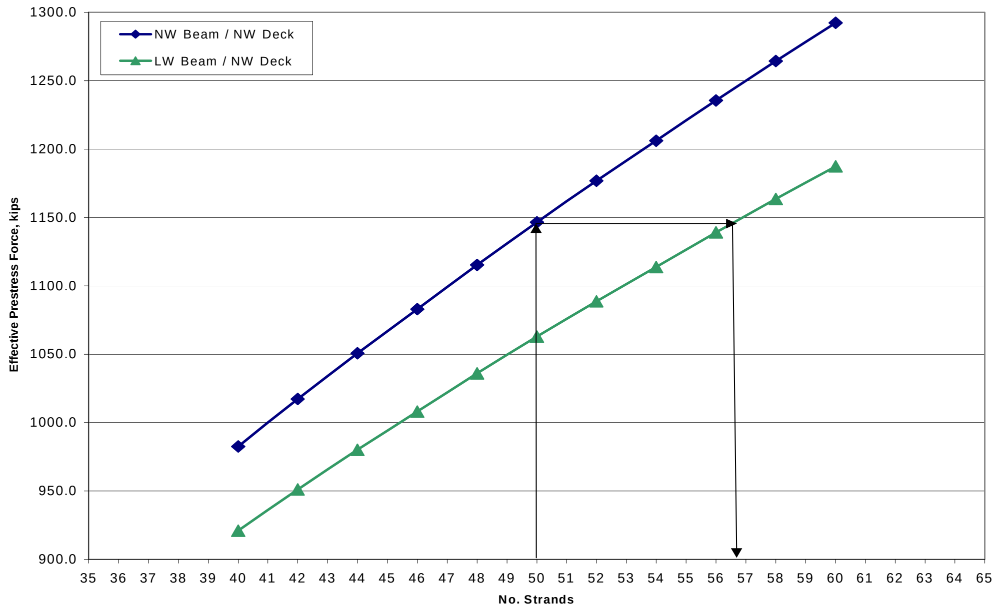


Figure B: Comparison of Effective Prestress Force

From this figure, it can be deduced a beam made from lightweight concrete will require a larger number of strands to maintain the same effective prestress force as the identical beam made from normal weight concrete. For instance, if 50 strands were required for a normal weight beam with a normal weight deck, approximately 58, rounding up to next increment of even strands, would be required for the lightweight section. This difference is of course again due to the higher prestress losses of the lightweight beam. This difference in strand requirements however does not explain why the sections noted above both require 50 strands. To further examine why the same number of strands are required for these sections, a comparison of the effective stresses for both beams that includes the self-weight of the members is required.

These comparisons of effective stresses, plotted at the bottom centerline of each beam, are shown in Figures C and D. The results of Figure C, which simply represents a plot of the effective stress for each of the beams, is as expected because it is basically another representation of the effective prestress force that was given in Figure B.

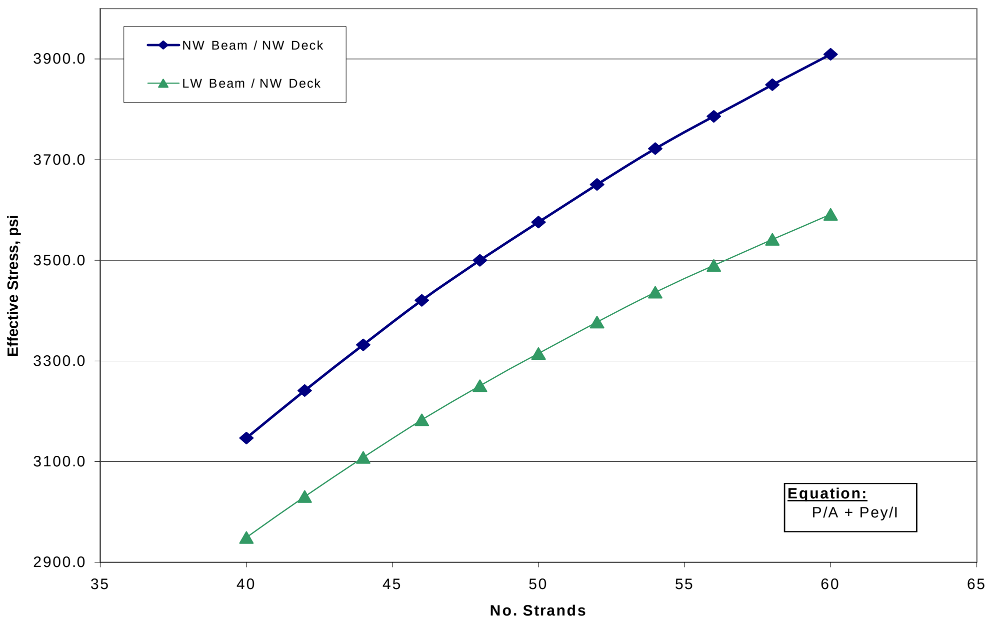


Figure C: Effective Stress at Bottom Centerline of Beam

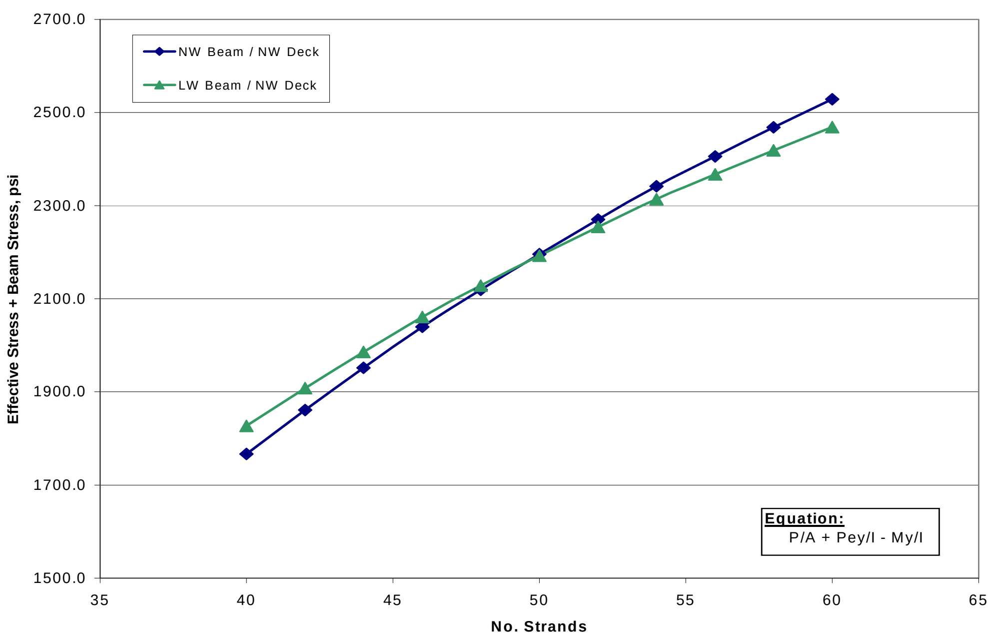


Figure D: Effective Stress Including Self-Weight at Bottom Centerline of Beam

In Figure D, the stress due to the self-weight of each beam is subtracted from the effective stress of the previous plot. As can be seen in this graph, the curves representing the stresses for each beam are almost coincident with each other and are certainly almost identical for 50 strands.

The final interpretation of this is that if the total external superimposed loads for each of the sections are considered equal, then the effective stress including self-weight for these sections are nearly equal. This is a result of the difference in prestress losses as well as the differences in stress due to the self-weight of the beams. This leads to the conclusion that the overall efficiency of the lightweight section considered in this case is negligible when compared to an equivalent normal weight section due mainly to the higher prestress losses. This highlights fact that the prestress losses play a very large role in the effectiveness and hence the efficiency of a beam made from lightweight concrete.

Because prestress losses in the lightweight concrete are crucial to the efficiency of this beam, the ACI method was used as a check on the prestress losses used in the analysis for these beams. This comparison is shown in Table B and it can be noted from this comparison that the prestress losses determined by each method are more similar for the lightweight concrete than the normal weight concrete. However, it must be considered that the differences would be much more pronounced if the maximum limits suggested by the ACI Method were used in the calculations. These maximum limits are 40,000 psi for normal weight concrete and 45,000 psi for lightweight concrete.

Table B: Comparison of Prestress Loss Methods

	Normal Weight		Lightweight	
	AASHTO (psi)	ACI (psi)	AASHTO (psi)	ACI (psi)
Shrinkage	9,500.0	8,220.0	9,500.0	8,220.0
Elastic Shortening	17,195.2	16,890.0	30,049.6	32,390.0
Creep	24,423.2	15,110.0	23,759.4	21,340.0
Steel Relaxation	1,584.3	3,390.0	332.1	2,660.0
Total:	52,702.7	43,610.0	63,641.1	64,610.0
% Difference in Totals	+21%		-1.5%	
Initial Prestress Loss	26.03%	21.50%	31.43%	30.20%
Final Prestress Loss	8.88%	9.18%	14.92%	16.70%

Note: Initial Prestress Loss was taken as ES + .5 CRs
ACI sets a max loss for normal weight concrete of 40,000 psi
ACI sets a max loss for lightweight concrete of 45,000 psi

In conclusion, the examination of two equivalent sections in which one section contained a normal weight beam and the other a lightweight beam has shown that the higher prestress losses for beams made from lightweight concrete reduce the beam's overall effectiveness. This causes the total effective stress including self-weight of the beams to be almost identical for the lightweight and normal weight beam, hence the lightweight beam for this scenario does not appear to have an advantage over a beam made from normal weight concrete.

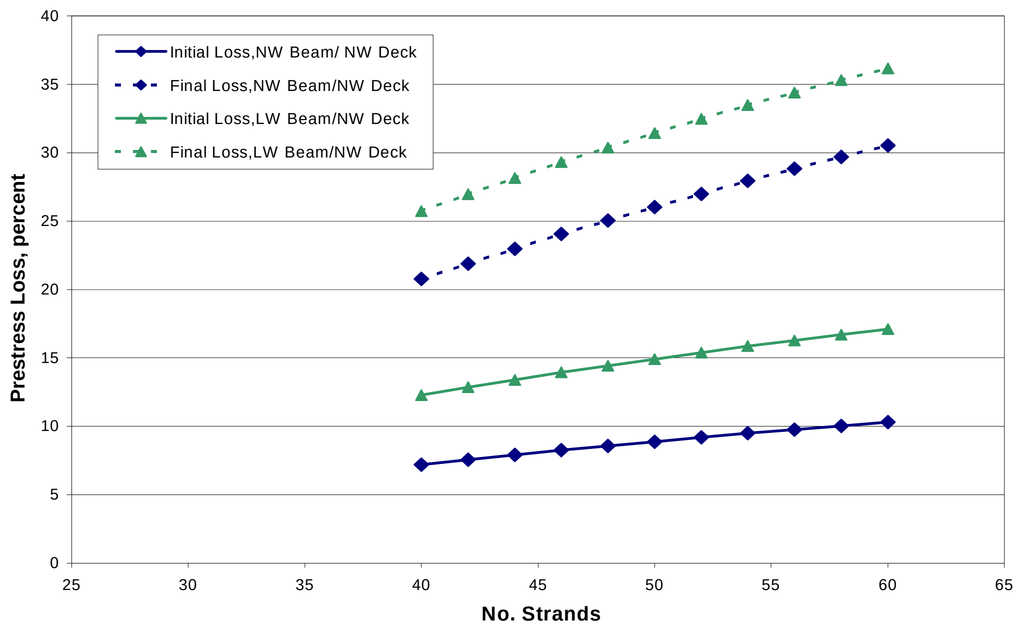
5.x Analysis Results

Table A below is a partial summary of the PSTRS14 analysis results obtained for the 7500 psi concrete beams. Comparing the results of the two highlighted sections in which one section consists of a normal weight beam and the other a lightweight beam, it is interesting to note that both require an equal number of prestressing strands. Intuitively, one would think that the lightweight beam would require fewer strands due to its lower density of 122 pcf. However, as will be shown in the figures that follow, higher prestress losses in this material counteract the dead load reduction and hence reduce the potential of material savings.

Table A: Partilal Summary of Prestressing Results for 7500 psi Beams

[redacted]

To gain a better understanding of the importance of the prestress losses for the lightweight beams, Figure A was developed. This figure depicts the variation of initial and final prestress losses as the number of prestressing strands for the normal and lightweight sections described above are varied between 40 and 60 strands.



<u>Figure A: Comparison of Prestress Losses</u>

From this figure, it is evident that the prestress losses are considerably higher for the lightweight beam. On average, the initial losses are approximately 68% higher for the lightweight as compared to the normal weight, while the final losses are approximately 21% higher. The higher losses for this beam can be attributed mostly to the lower modulus of elasticity that is usually typical of the lightweight concrete.

The higher prestress losses determined for the lightweight concrete beam translate directly to a lower effective prestress force for this member as determined by Equation 5.1 and as shown in Figure B. This is not surprising considering the fact that the effective prestress force is directly proportional to the loss of prestress.

$$.75 \times f'_s \times A^*_s \times N \times \Delta f_s \qquad \text{(Equation 5.1)}$$

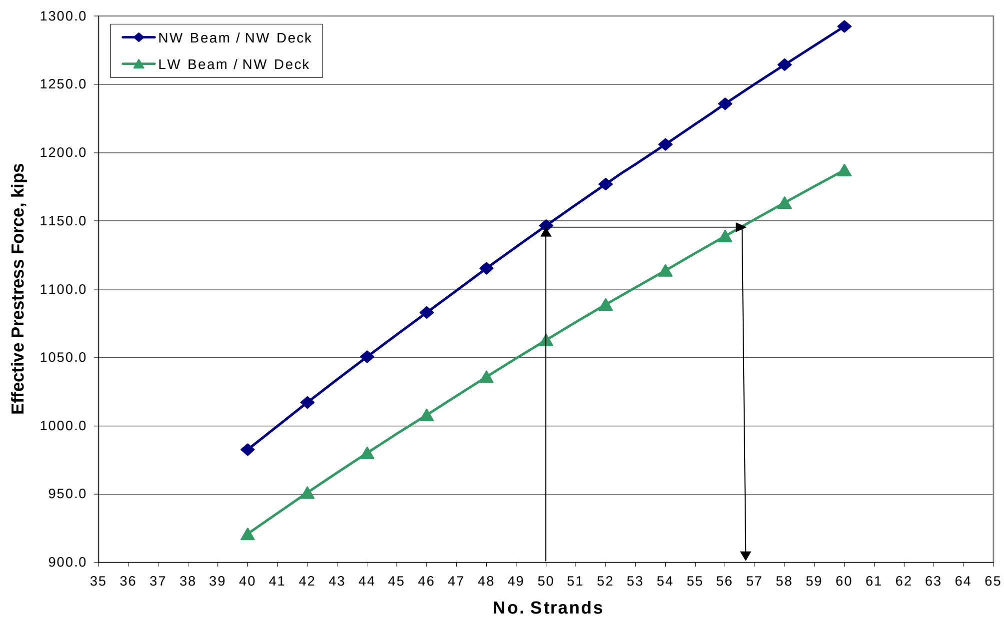


Figure B: Comparison of Effective Prestress Force

This figure can also be used to determine the number of strands that would be required for a lightweight beam to maintain the same prestress force as a beam made from normal weight concrete. As an example, if 50 strands were required for a normal weight beam with a normal weight deck, approximately 58 strands, rounding up to next even increment, would be required for the lightweight section (shown by lines with arrows). This difference is of course again due to the higher prestress losses of the lightweight beam. This difference in strand requirements however does not directly justify why the sections being compared both require

50 strands. To further examine why the same numbers of strands are required, a comparison of the effective stresses for both beams that include the self-weight of the members is necessary.

Figures C and D both on the following page, present these comparisons of effective stress for the normal and lightweight beam. Figure C simply shows the effective stress and really does not offer any new information. However, Figure D is the effective stress taking into account the stress induced by the self-weight of the members. From this figure, it is evident that the curves for the effective stress of these two beams become almost coincident with each other. This indicates that the difference in prestress losses in combination with the difference in self-weight cause these two members to experience almost the same stress, and almost exactly if the members each have 50 strands.

The final interpretation of this is that if the total external superimposed loads for each of the sections are considered equal, then the effective stress including self-weight for these sections are nearly equal. This is a result of the difference in prestress losses as well as the differences in stress due to the density of the beams. This leads to the conclusion that the overall efficiency of the lightweight section due to the lower density can be considered in this case to be negligible when compared to a similar normal weight section. This highlights the fact that the prestress losses play a very large role in the effectiveness and hence the efficiency of a beam made from lightweight concrete.

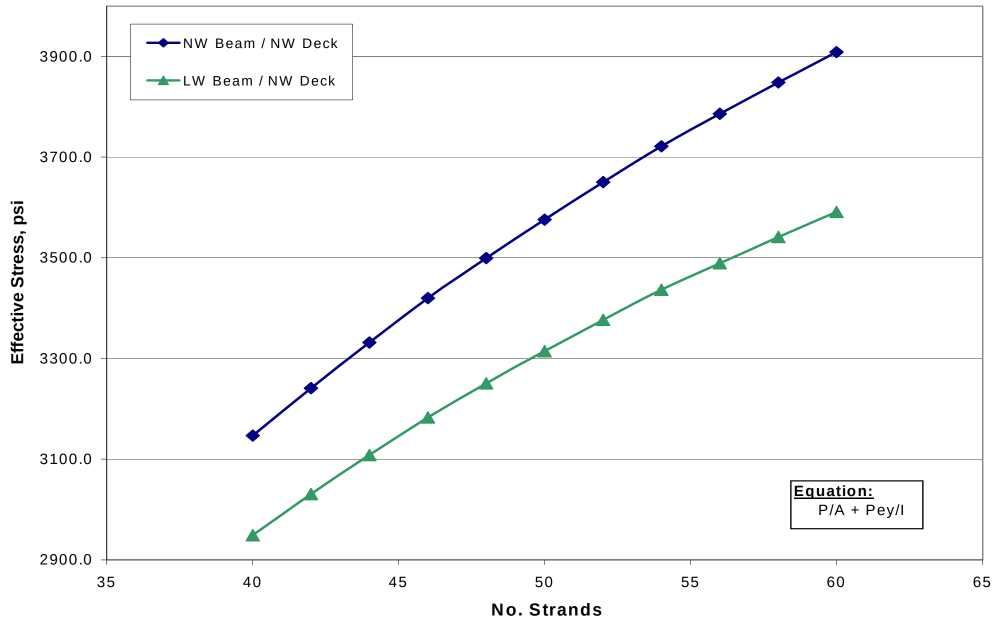


Figure C: Effective Stress at Bottom Centerline of Beam

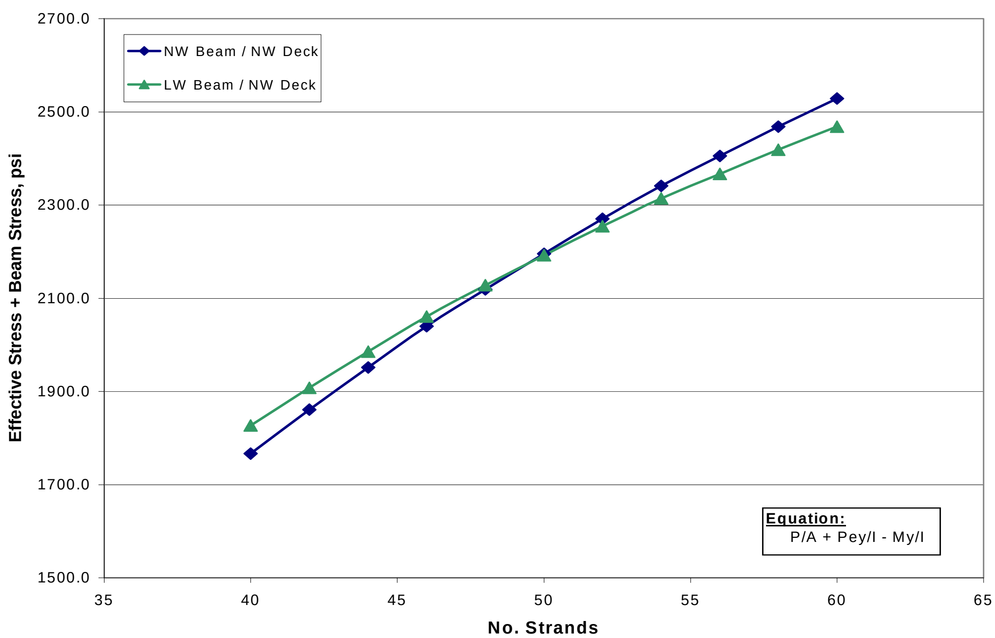


Figure D: Effective Stress Including Self-Weight at Bottom Centerline of Beam

Prestress losses in this analysis were determined using the AASHTO method, but because these losses in the lightweight concrete are crucial to the efficiency of the lightweight beam, the ACI method was used as comparison check. This comparison is shown in Table B and it can be noted that the prestress losses determined by each method are more similar for the lightweight concrete than the normal weight concrete. However, it must be considered that the differences would be much more pronounced if the maximum limits suggested by the ACI Method were used in the calculations. These maximum limits are 40,000 psi for normal weight concrete and 45,000 psi for lightweight concrete. It must also be emphasized that the greatest difference in the losses between in the normal and lightweight concrete is due to elastic shortening which is inversely proportional to the initial modulus of elasticity.

Table B: Comparison of Prestress Loss Methods

	Normal Weight		Lightweight	
	AASHTO (psi)	ACI (psi)	AASHTO (psi)	ACI (psi)
Shrinkage	9,500.0	8,220.0	9,500.0	8,220.0
Elastic Shortening	17,195.2	16,890.0	30,049.6	32,390.0
Creep	24,423.2	15,110.0	23,759.4	21,340.0
Steel Relaxation	1,584.3	3,390.0	332.1	2,660.0
Total:	52,702.7	43,610.0	63,641.1	64,610.0
% Difference in Totals	+21%		-1.5%	

Initial Prestress Loss	26.03%	21.50%	31.43%	30.20%
Final Prestress Loss	8.88%	9.18%	14.92%	16.70%

Note: Initial Prestress Loss was taken as ES + .5 CRs
ACI sets a max loss for normal weight concrete of 40,000 psi
ACI sets a max loss for lightweight concrete of 45,000 psi

In conclusion, the examination of two equivalent sections with one section consisting of a normal weight beam and the other a lightweight beam, both with normal weight decks, has shown that the higher prestress losses for beams made from lightweight concrete reduce the beam's overall effectiveness. This causes the total effective stress including self-weight of the beams to be almost identical for these beams; hence the lightweight beam for this scenario does not appear to have an advantage over a beam made from normal weight concrete. A possible alternative to overcoming the elastic shortening losses that are crucial due to the low initial elastic modulus of the lightweight concrete would be in a post-tensioned application. It may be possible that this type of application would take greater advantage of this material's low density and offer larger potential for material savings.

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