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16. Abstract

This is the third of four reports that document the findings of a Texas Department of Transportation sponsored project to evaluate the allowable stresses and resistance factors for high strength concrete (HSC) prestressed bridge girders. The second phase of this research study, which is documented in this volume, focused on three major objectives: (1) to determine the current state of practice for the design of HSC prestressed bridge girders, (2) to evaluate the controlling limit states for the design of HSC prestressed bridge girders and identify areas where some economy in design may be gained, and (3) to conduct a preliminary assessment of the impact of raising critical flexural design criteria with an objective of increasing the economy and potential span length of HSC prestressed girders.

The first objective was accomplished through a literature search and survey. The literature search included review of design criteria for both the American Association of State and Highway Transportation Officials (AASHTO) Standard and Load and Resistance Factor Design (LRFD) Specifications. Review of relevant case studies of the performance of HSC prestressed bridge girders as well as important design parameters for HSC were carried out. In addition, researchers conducted a survey to gather information and document critical aspects of current design practices for HSC prestressed bridges. The second objective was accomplished by conducting a parametric study for single-span HSC prestressed bridge girders to primarily investigate the controlling flexural limit states for both the AASHTO Standard and LRFD Specifications. AASHTO Type IV and Texas U54 girder sections were considered. The effects of changes in concrete strength, strand diameter, girder spacing, and span length were evaluated. Based on the results from the parametric study, the limiting design criteria for HSC prestressed U54 and Type IV girders using both the AASHTO Standard and LRFD Specifications for Highway Bridges were evaluated. Critical areas where some economy in design may be gained were identified. The third research objective was accomplished by evaluating the impact of raising the allowable tensile stress for service conditions. The stress limit selected for further study was based on the current limit for uncracked sections provided by the American Concrete Institute (ACI) 318-02 building code and the limit used for a specific case study bridge in Texas. Recommendations for improving some critical areas of current bridge designs and for increasing bridge span lengths are given.

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FLEXURAL DESIGN OF HIGH STRENGTH CONCRETE PRESTRESSED BRIDGE GIRDERS – REVIEW OF CURRENT PRACTICE AND PARAMETRIC STUDY

by

Mary Beth D. Hueste, P.E.
Assistant Research Engineer
Texas Transportation Institute

and

Gladys G. Cuadros
Graduate Research Assistant
Texas Transportation Institute

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The Texas A&M University System
College Station, Texas 77843-3135

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TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

TABLE Page

TABLE Page

TABLE Page

TABLE Page

TABLE Page

TABLE Page

TABLE Page

TABLE Page

TABLE Page

TABLE Page

1 INTRODUCTION

1.1 BACKGROUND AND PROBLEM STATEMENT

Over the years, design procedures for engineered structures have been developed to provide satisfactory margins of safety. Engineers based these procedures on their confidence in the analysis of the load effects and the strength of the materials provided. As analysis techniques and quality control for materials improve, the design procedures change. Current research and changes in design practices for bridges tend to focus on the American Association of State and Highway Transportation Officials (AASHTO) Standard and Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO 2002b).

The AASHTO Standard Specifications for Highway Bridges (AASHTO 2002a) use both allowable stress design (ASD) and load factor design (LFD) philosophies. However, the AASHTO LRFD Specifications, referred to as load and resistance factor design (LRFD), are written in a probability-based limit state format. In this case, safety against structural failure is quantified using reliability theory, where the selection of conservative load and resistance factors take into account the statistical variability of the design parameters. Load and resistance factors are determined for each ultimate limit state considered, and safety is measured in terms of the target reliability index (Nowak and Collins 2000). As a result, the LRFD Specifications allow for a more uniform safety level for various groups of bridges for the ultimate limit states. However, for prestressed concrete design, traditional serviceability limit states are still used and often control the flexural design of prestressed concrete bridge girders.

On the other hand, as technology has improved throughout the last decade, the development of high strength concrete (HSC) has progressed at a considerable rate. Concrete strengths up to 12000 psi or more can be obtained through the optimization of concrete mixture proportions, materials, and admixtures. Despite this trend, bridge designers have been cautious to specify HSC for their precast, prestressed concrete designs, and the application of HSC has been limited primarily to important high-rise buildings. This reluctance is understandable given the empirical nature of the design equations provided by the AASHTO codes for prestressed concrete members, as well as the fact that these formulas were developed based on the mechanical properties of normal strength concrete (NSC) of 6000 psi or less.

Highway bridge demands often result in the need for longer spans, fewer girders, and consequently, fewer piers and foundations. The use of HSC prestressed bridge girders, along with appropriate design criteria, would enable designers to utilize HSC to its full potential. This would result in several practical advantages. Ralls (1985) anticipates longer span beams that are cost-effective at the time of construction and during the life of the structures. Therefore, more data on statistical parameters for the mechanical properties of HSC (more than 6000 psi) along with identification of critical areas for refining current design provisions for HSC prestressed bridge girders are needed to fully utilize HSC.

1.2 OBJECTIVES AND SCOPE

This report summarizes Phase 2 of the Texas Department of Transportation (TxDOT) Research Project 0-2101, “Allowable Stresses and Resistance Factors for High Strength Concrete.” The objective of this project was to evaluate the allowable stresses and resistance factors in the AASHTO LRFD Specifications for design of HSC girders used in Texas bridges. Hueste et al. (2003a) summarized the complete project. Phase 1 of this project (Hueste et al. 2003b) evaluated the applicability of current prediction equations for estimation of mechanical properties of HSC and determined statistical parameters for mechanical properties of HSC. The HSC samples for Phase 1 were collected from three Texas precasters that manufacture HSC prestressed bridge girders. Phase 3 of this project assessed the impact of different curing conditions on the compressive and flexural strength of HSC mixtures used for prestressed girders in Texas (Hueste et al. 2003c). The portion of the research project addressed by this report (Phase 2) includes defining the current state of practice for design of HSC prestressed girders and identifying critical design parameters that limit the design of typical HSC prestressed bridge girders. There are three specific objectives for this study:

1. determine the current state of practice for HSC prestressed bridge girders across the United States,
2. evaluate the controlling limit states for the design of HSC prestressed bridge girders and identify areas where some economy in design may be gained, and
3. conduct a preliminary assessment of the impact of revising critical design criteria with an objective of increasing the economy of HSC prestressed girders.

1.3 RESEARCH PLAN

In order to accomplish these objectives, researchers performed the following major tasks.

Task 1: Review Previous Research and Current State of Practice

A literature review was conducted to document the current state of practice of prestressed concrete bridge girders, including review of design criteria and relevant case studies of the performance of HSC prestressed bridge girders. In addition to the literature search, a survey was developed and distributed to all 50 state departments of transportation as well as to several organizations involved in the design of bridge structures. The objective of this survey is to gather information and document critical aspects of current design practices for HSC prestressed bridge girders.

Task 2: Comparison of Design Provisions for Prestressed Concrete Bridge Girders

The main purpose of this task is to provide background information and a comparison of the current AASHTO LRFD and Standard Specifications for prestressed concrete bridge girders. Task 2 outlines the differences in the design philosophy and calculation procedures for these two specifications.

Task 3: Parametric Study

A parametric study was conducted for single-span prestressed concrete bridge girders mainly to investigate the controlling limit states for both the AASHTO Standard and LRFD Specifications for Highway Bridges. Both Type IV and U54 girder sections were evaluated, with consideration given to the effects of changes in concrete strength, strand diameter, girder spacing, and span length.

Task 4: Evaluation of the Controlling Limit States for HSC Prestressed Bridge Girders

The purpose of this task is to evaluate the limiting design criteria for HSC prestressed U54 and Type IV beams using both the AASHTO Standard and LRFD Specifications for Highway Bridges. This evaluation uses results from the parametric study. The potential impact of revised design criteria was also evaluated.

Task 5: Develop Summary, Conclusions, and Recommendations

This task includes a summary of work accomplished, description of findings, conclusions, and recommendations. Critical areas for refining current design provisions for HSC prestressed bridge girders are identified.

1.4 OUTLINE OF THIS REPORT

This report is organized as follows. Chapter 1 provides an introduction to the project. Chapter 2 provides a review of previous research related to HSC prestressed bridge girders. Chapter 3 provides a review of current specifications and practices for the design of prestressed concrete bridge girders, along with applicable design documents used by TxDOT. Chapter 4 describes the results of the survey to document relevant aspects of current practice for the design of HSC prestressed bridge girders. Chapter 5 outlines a parametric study for the Texas U54 and AASHTO Type IV beams to mainly evaluate the controlling limit states for the design of HSC prestressed bridge girders. Chapters 6 and 7 evaluate the results of the parametric study for the U54 and Type IV beams, respectively, along with an assessment of the impact of potential revisions to design criteria. Finally, Chapter 8 provides a summary of the project, conclusions, and recommendations for future research. Additional information such as the questionnaire for the survey, live load distribution factors and moments for the Standard and LRFD Specifications, and complete designs for the U54 and Type IV beams are presented in the appendices.

2 PREVIOUS RESEARCH

2.1 GENERAL

Several studies have evaluated the use of HSC for prestressed bridge girders. Topics of importance to this project, which are reviewed in this chapter, include the use of HSC for prestressed bridge girders, flexural design of prestressed concrete bridge girders, development of the AASHTO LRFD Specifications, allowable stress limits for prestressed concrete beams, critical mechanical properties of HSC for design, and concrete strengths at transfer.

2.2 USE OF HSC FOR PRESTRESSED BRIDGE GIRDERS

2.2.1 Impact of HSC

Durning and Rear (1993) assessed the viability and performance of HSC for Texas bridge girders. Results showed that for AASHTO Type C and Type IV girders with a girder spacing of approximately 8.4 ft., an increase in concrete compressive strengths from 6000 to 10000 psi results in approximately a 20 percent increase in the maximum span lengths. Type IV girders with 0.5 in. diameter strands can fully utilize concrete compressive strengths up to 10000 psi. Therefore, to effectively use higher concrete strengths (above 10000 psi), 0.6 in. diameter strands should be used. They also found that when using HSC with 0.6 in. diameter strands, longer span lengths can be reached and the girder spacing can be doubled for a given span length. This reduces the number of girders in a bridge. Consequently, fewer piers and foundations are required, resulting in substantial savings.

Russell (1994) reported that an increase in compressive strength from 6000 to 10000 psi results in a 25 percent increase in span capacity for AASHTO Type IV girders and a 21 percent increase in span capacity for Texas U54 girders when 0.5 in. diameter strands are used.

Adelman and Cousins (1990) evaluated the use of HSC bridge girders in Louisiana. They found that an increase in concrete compressive strength from 6000 to 10000 psi results in a 10 percent average increase in span capacity for seven types of girders using 0.5 in. diameter strands. In particular, an average of a 12 percent increase in span capacity for the AASHTO Type IV girder, which included several girder spacings, was reported.

2.2.2 Example Structures

A description of two Texas bridges constructed with HSC prestressed girders is given below to provide important applications and relevant background of such bridges.

2.2.2.1 Louetta Road Overpass, State Highway 249, Houston, Texas

The Louetta Road Overpass is a high performance concrete (HPC) bridge constructed in 1995 as a part of a research project conducted by TxDOT in cooperation with the University of Texas at Austin. The benefits of the use of HSC in combination with HPC for the girder design allowed for a simple span construction. In addition, the bridge met aesthetic considerations since it used a reduced number of beams and piers. HPC was used not only because high concrete strength was required but also because placement of the concrete in the U-beam formwork was necessary. Thus, more workability was required and a set retarder and high-range water-reducing admixture was used. No accelerated curing was used; cement was partially replaced with fly ash.

The span length of the bridge is 130 ft. with precast pretensioned U54 Beams and precast panels with a cast-in-place (CIP) topping slab. The required concrete strengths at service (at 56 days) were from 10000 to 13000 psi. Transfer (16-21 hours) concrete strengths were from 6900 to 8800 psi. These strengths varied according to the requirement for each particular beam. The prestressing consisted of 0.6 in. diameter strands on a 1.97 in. grid spacing, with a total of 87 strands. The maximum debonding length was 30 ft., which is an exception to the typical maximum debonding length of 20 ft. (Ralls 1995). Designers used a maximum allowable tensile stress at transfer of $10\sqrt{f'_{ci}}$ rather than the code limit of $7.5\sqrt{f'_{ci}}$ (where f'_{ci} is in psi units). An allowable tensile stress at service of $8\sqrt{f'_c}$ rather than the code limit of $6\sqrt{f'_c}$ for 28 days was used for design (where f'_c is in psi units). Testing of the actual concrete mix showed that these values are adequate (Ralls 1995).

2.2.2.2 San Angelo Bridge, U.S. Route 67, San Angelo, Texas

The San Angelo Bridge is an HPC bridge recently constructed by TxDOT (from 1995 to 1998). HPC was used because not only HSC was required but also placement of the concrete in the I-beam was necessary (see Section 5 for geometry). Thus, TxDOT used a set retarder and high-range water-reducing admixture. No accelerated curing was used, and cement was partially replaced with fly ash.

The span length of the bridge is 153 ft., and the girders are precast pretensioned Type IV beams using precast panels with a CIP topping slab. The required concrete strengths at service (at 56 days) were from 5800 to 14700 psi. Transfer (16-21 hours) concrete strengths were from 8900 to 10800 psi. These strengths varied according to the requirements for each particular beam. The prestressing consisted of 0.6 in diameter strands on a 2 in. grid spacing. Again, for this bridge the benefits of the use of HSC in combination with HPC in the girder design allowed for a simple span construction, and aesthetic considerations were met because fewer beams and piers were required.

2.3 FLEXURAL DESIGN OF PRESTRESSED CONCRETE BRIDGE GIRDERS

2.3.1 Design Procedure

The basic flexural design procedure for prestressed concrete bridge girders is similar for both the AASHTO Standard and LRFD Specifications. The traditional process consists of first satisfying serviceability conditions and then checking the ultimate limit state. For flexure, the required serviceability conditions to be checked consist of ensuring that the flexural stresses do not exceed the allowable stresses at critical load stages. The ultimate state to be checked for flexure involves verifying that the factored moment demand does not exceed the reduced nominal moment strength. Current designs for prestressed concrete girders are typically governed by the allowable stress requirements. The LRFD Specifications were calibrated assuming that the maximum design load effect governs designs, and the load and resistance factors were determined for ultimate conditions (Nowak 1999). The LRFD Specifications also provide limit state design rules (Service I, Service III, and Strength I) for design of prestressed concrete that only work consistently with the LRFD philosophy at the ultimate limit states (Strength I). Additional details for the design of prestressed concrete bridge girders using both the AASHTO Standard and the LRFD Specifications are provided in Sections 3 and 5.

2.3.2 Current Specifications

As of 2002, AASHTO had issued two design specifications for highway bridges: the AASHTO Standard Specifications for Highway Bridges, 16th Edition and 2002 Interim Revisions, and the AASHTO LRFD Bridge Design Specifications, 2nd Edition and 2002 Interim Revisions (AASHTO 2002 a,b). In 2003, the Standard Specifications for Highway Bridges, 17th Edition, was released (AASHTO 2003). This project references AASHTO (2002 a,b).

The AASHTO Standard Specifications use the ASD and the load LFD philosophies. However, the AASHTO LRFD Specifications, referred to as load and resistance factor design, are written in a probability-based limit state format where safety is provided through the selection of conservative load and resistance factors. The LRFD specifications determine load and resistance factors for each limit state considered and measure safety in terms of the target reliability index (Nowak and Collins 2000). Unlike the Standard Specifications, the calibration of the LRFD Specifications is based on reliability theory and allows for designs with a more uniform level of safety.

Research discussed in Section 4 indicates that the departments of transportation in the United States are moving toward using the new LRFD Specifications, although this transition is gradual. The Standard Specifications are still widely used. Most states plan complete implementation of the LRFD Specifications in the period of 2004 to 2007 (Section 4.2.1).

Three main reasons can be identified to explain the preference for the Standard Specifications:

- LRFD uses a new probability-based limit state format that designers are still reluctant to use.
- Some studies indicate that the choice of design specifications has little impact on the span capabilities for a given type of beam.
- Experience has shown that bridges designed under the Standard Specifications are performing as expected and most of them have worked well.

Some important differences exist between the flexural design provisions for the AASHTO Standard and LRFD Specifications. The significant changes in the LRFD Specifications include the introduction of a new live load model, a new impact load factor, new live load distribution factors, as well as changes in the description of the limit states. Additional information is provided in Section 3.

2.4 DEVELOPMENT OF THE AASHTO LRFD SPECIFICATIONS

To show the importance of the statistics and parameters of resistance, this section summarizes the calibration procedure for the AASHTO LRFD Specifications. Load and resistance factors are determined for the ultimate limit state, and safety is measured in terms of the target reliability index (β_T), which allows for a uniform and acceptably low probability of failure (p_f) for various groups of bridge girders. Relevant aspects of the calibration procedure are the choice of the load and resistance statistical parameters as well as the target reliability index. It should be noted that the statistical parameters for resistance of concrete members used in the code calibration are based on mechanical properties for NSC. Phase 1 of this study determined statistical parameters for HSC produced by Texas precasters (Hueste et al. 2003b).

The AASHTO LRFD Specifications were calibrated to provide design provisions for steel girder bridges (composite and non-composite), reinforced concrete bridges (T-beams), and prestressed concrete bridges (AASHTO girders) (Nowak 1999). The design provisions were developed for the ultimate limit state. However, there is still a need to consider the allowable stress design since serviceability limit states often govern the flexural design of prestressed concrete bridge girders. Therefore, both the serviceability limit state (SLS) and the ultimate limit state (ULS) prescribed by LRFD should be considered in the flexural design of prestressed concrete bridge girders.

The objective of the calibration process for the LRFD Specifications was to select a set of values for the load and resistance factors that would provide a uniform safety level in design situations covered by the code. The required safety level was defined by a target reliability index (β_T.) The target reliability index for the ULS was taken as β_T = 3.5. Although many combinations of load and resistance factors can be used to attain the target reliability index, it is desirable to have the same load factor for each load type for different types of construction. Nowak (1999) summarized the procedure for calibration of the AASHTO LRFD Specifications as follows.

1. *Development of a database of sample current bridges*

   Approximately 200 bridges were selected from various regions of the United States. The selection was based on structural type, material, and geographic location. Future trends were considered by sending questionnaires to various departments of transportation. For each bridge in the database, the loads indicated by the contract drawings were subdivided by the following weights: factory-made elements, cast-in-place concrete members, wearing surface, miscellaneous (railing, luminaries), HS20 live load, and dynamic loads.

2. *Development of a set of bridge designs for calculation purposes*

   A simulated set of 175 bridge designs was developed based on the relative amount for the loads identified above for each type of bridge, span, and girder spacing in the database.

3. *Establishment of the statistical database for load and resistance parameters*

   Because the reliability indices are computed in terms of the mean and standard deviations of load and resistance, determination of these statistical parameters was very important. Statistical parameters of load and resistance were determined on the basis of the available data, such as truck surveys and material testing, and by simulations.

4. *Estimation of the reliability indices implicit in the current design*

It was assumed that the total load (Q) is a normal random variable and the resistance (R) is a log-normal random variable. The Rackwitz and Fiessler (1978) method was used to compute the reliability indices, β. This method is an iterative procedure based on normal approximations to non-normal distributions at a design point. For simplicity the method uses only two random variables: the resistance, R, and total load effects, Q. The mean (m_Q) and standard deviation (σ_Q) of Q were calculated using Turkstra's rule (Nowak and Collins 2000), and the resistance parameters bias (λ_R) and covariance (V_R) were calculated using Monte Carlo simulation. Once the resistance parameters (R_n, λ_R, and V_R) and the load parameters (m_Q and σ_Q) were determined, the reliability indices were calculated for each type of bridge girder for the moment and shear limit state. R was computed using the equation from the AASHTO Standard Specifications: $[1.3D + 2.17(L+I)]/\phi$, where D corresponds to the dead load demand and $L+I$ corresponds to the live load plus impact. Also, the resistance factors (ϕ) were from the AASHTO Standard Specifications.

5. *Selection of the target reliability index*

Reliability indices were calculated for each simulated bridge for both moment and shear. The results gave a wide range of values for the reliability indices resulting from this phase of the calibration process. However, this was expected since the designs were based on the AASHTO Standard Specifications. From these calculated reliability indices and from past calibration of other specifications, a target reliability index $\beta_T = 3.5$ was chosen.

The most important parameters that determine the reliability index are span length and girder spacing. In calibrating the LRFD Specifications, the corresponding safety level of 3.5 determined for a simple span moment, corresponding to girder spacing of 6 ft. and span of 60 ft. was considered acceptable. The reliability index is a comparative indicator, where a group of bridges having a reliability index greater than a second group is safer ($\beta = \phi^{-1}[p_f]$, where p_f is the probability of failure).

6. *Computation of the load and resistance factors*

To achieve a uniform safety level for all materials, spans, and girder spacings, the load and resistance factors were determined. One way to find the load and resistance factors is to select the load factors and then calculate the resistance factors, as follows:

- Factored load was defined as the average value of load, plus some number of standard deviations (k) of the load: $\gamma_i = \lambda_i (1 + kV_i)$.
- For a given set of load factors, the value of the resistance factors can be assumed. The corresponding reliability index is computed and compared with the target reliability index (resistance factors are rounded to 0.05). If the values are close, a suitable combination of load and resistance factors has found.
- If close values do not result, a new trial set of load factors has to be used and the process is repeated until the reliability index is close to the target value.

After studies were conducted, a value for k = 2 was recommended. For prestressed concrete bridge girders, values of ϕ = 1.0 for the moment limit state and ϕ = 0.95 for the shear limit state were found. Recommended values of load factors corresponding to k = 2 are: $[1.25D + 1.5D_A + 1.7(L+I)]/\phi$, where D corresponds to the dead load demand, D_A is the weight of the asphalt, and $L+I$ corresponds to the live load plus impact.

7. *Computation of reliability indices*

Finally, reliability indices were computed for designs found considering the new calibrated LRFD load and resistance factors. Results were plotted, and they showed that the new reliability indices closely matched the target reliability index (Nowak 1999).

2.5 ALLOWABLE STRESS LIMITS FOR PRESTRESSED CONCRETE BEAMS

The design of prestressed concrete members is typically governed by the service condition where the flexural stresses at various load stages are limited to the corresponding allowable stresses. Therefore, these allowable stresses are an indicator of the resistance and these limits become more critical for designs using HSC. Current specifications provide allowable stress limits that were developed based on the mechanical properties of NSC of 6000 psi or less. These allowable stresses that traditionally are conservative for standard designs using NSC may not be appropriate for HSC designs. Therefore, current allowable stresses need to be reviewed and, if appropriate, revised based on information from testing of HSC (see Hueste et al. 2003 a,b,d).

In this study, one of the objectives was to perform a preliminary assessment of the impact of raising critical design criteria with the objective of increasing the economy of HSC prestressed girders. Therefore, researchers studied the limit states that control the required number of strands and, consequently, the span capacity were studied.

Allowable stresses for concrete specified by both the AASHTO Standard and LRFD Specifications are shown in Table 2.1. Both specifications provide almost the same allowable stresses except for the compressive stress at service, which was increased from 0.4 f'_c to 0.45 f'_c in the LRFD Specifications. Maximum limits for the tensile stress at transfer are slightly different (see note for Table 2.1). On the other hand, the American Concrete Institute (ACI) 318-02 building code (ACI Comm. 318 2002) provides serviceability requirements that are also slightly different than those provided by the AASHTO Standard and LRFD Specifications. Table 2.2 shows these limits.

In ACI 318-02, the allowable tensile stress at service is limited to $7.5\sqrt{f'_c}$. This limit results in an allowable tensile stress that is 25 percent greater than the corresponding limiting stress given by both the AASHTO Standard and LRFD Specifications ($6\sqrt{f'_c}$). However, ACI 318-02 specifies the same allowable compressive stress as the LRFD Specifications. In ACI 318-02, the maximum allowable tensile stress at transfer is 25 percent lower than that given by the AASHTO Standard or LRFD Specifications for the case when bonded reinforcement is provided. ACI 318-02 states that the tensile stress in concrete immediately after prestress transfer shall not exceed $3\sqrt{f'_{ci}}$ except when the stress is computed at the end beams of simply supported members, where this value can go up to $6\sqrt{f'_{ci}}$ ($7.5\sqrt{f'_{ci}}$ for the AASHTO Standard and LRFD Specifications). If the tensile stress exceeds this value, bonded auxiliary reinforcement must be provided in the tensile zone.

Table 2.1. Allowable Stresses Specified by the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b).

Type of Stress		Allowable Stress (psi)
Initial Stage: Immediately after Transfer (After Initial Loss in the Prestressing Force)	Tension	200 or $3\sqrt{f'_{ci}}$ *
	Compression	$0.6f'_c$
Intermediate Stage: After Cast-in-Place Concrete Slab Hardens. Only Sustained Loads. (After Final Loss in the Prestressing Force)	Compression	Standard: $0.4f'_c$ LRFD: $0.45f'_c$
Final Stage: Total Dead and Live Loads (After Final Loss in the Prestressing Force)	Tension	$6\sqrt{f'_c}$
	Compression	$0.6f'_c$
Additional Check of the Compressive Stress at the Final Stage	Compression	$0.4f'_c$

Note:
* When the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete. The maximum tensile stress at transfer shall not exceed $7.5\sqrt{f'_{ci}}$ for the AASHTO Standard Specifications and $6.957\sqrt{f'_{ci}}$ for the LRFD Specifications ($0.22\sqrt{f'_{ci}}$ in ksi) when bonded reinforcement is provided.

Table 2.2. Allowable Stresses Specified by ACI 318-02.

Type of Stress		Allowable Stress (psi)
Initial Stage: Immediately after Transfer (After Initial Loss in the Prestressing Force)	Tension	200 or $3\sqrt{f'_{ci}}$ *
	Compression	$0.6f'_c$
Intermediate Stage: After Cast-in-Place Concrete Slab Hardens. Only Sustained Loads. (After Final Loss in the Prestressing Force)	Compression	$0.45f'_c$
Final Stage: Total Dead and Live Loads (After Final Loss in the Prestressing Force)	Tension	$7.5\sqrt{f'_c}$
	Compression	$0.6f'_c$

Note:
* The tensile stress can exceed this value when the stress is computed at the end beams of simply supported members and can go up to $6\sqrt{f'_{ci}}$. If the tensile stress exceeds the limiting value in the table, bonded reinforcement shall be provided to resist the total tension force in the concrete.

In regard to the tensile stress limit, T.Y. Lin (1963) stated the following:

> "What should be the tensile stress in continuous prestressed concrete beams at the point of cracking? Some engineers believe that the cracking tensile strength is higher than the modulus of rupture measured from plain concrete strength specimens. Experience however has shown that the modulus of rupture is a reasonably accurate measure of the start of cracking in continuous prestressed beams. Before the start of actual cracking, some plastic deformation is usually exhibited in the concrete. Such deformations occur only in limit regions and do not affect the general behavior of the structure as an elastic body. Hence, the validity of the elastic theory can still be counted on, up to and perhaps slightly beyond the cracking of concrete."

Some basis for the compressive stress limit were found to explain the reasons for the coefficients of the limits currently given in the code. T.Y. Lin (1963, p. 525) stated the following:

> "At jacking or at transfer of prestress the amount of prestress is rather accurately known, there is little likelihood of excessive loading, and there is no danger of fatigue; hence a relatively high stress is permissible. Most codes set 0.6 f'_c as the maximum allowable compressive stress. Here the stress is not controlled by the consideration of overload capacities but rather by the possibility of excessive creep, camber, or other local strains."

> "When considering the structure under service loads, there is a possibility of fatigue effect and occasional excessive overloads; hence lower values must be allowed. Generally 0.4 f'_c for bridges and 0.45 f'_c for buildings are the maximum. Higher values can be justified only after careful investigations of fatigue and ultimate strength. The value of f'_c is usually based on the 28-day strength but may occasionally be based on the strength of concrete at the time of service if such strength can be assured. Depending on the shape of the section, the above allowable value will usually yield a factor of safety of 2.5 to 3, which is ample. Occasionally, a factor of safety of only about 2 is attained, which seems inadequate for concrete except where overloads and repeated loading are not at all likely."

The commentary for ACI 318-02 (ACI Comm. 318 2002, p. 266) provides the following explanation for the compressive stress limit:

> "The compressive stress limit of 0.45 f'_c was conservatively established to decrease the probability of failure of prestressed concrete members due to repeated loads. This limit seemed reasonable to preclude excessive creep deformation. At higher values of stress, creep strains tend to increase more rapidly as applied stress increases.

> The change in allowable stress in the 1995 code recognized that fatigue tests of prestressed concrete beams have shown that concrete failures are not the controlling criterion. Designs with transient live loads that are large compared to sustained live and dead loads have been penalized by the previous single compression stress limit. Therefore, the stress limit of 0.6 f'_c permits a one-third increase in allowable compression stress for members subject to transient loads.
>
> Sustained live load is any portion of the service live load that will be sustained for a sufficient period to cause significantly time-dependent deflections. Thus, when the sustained live and dead loads are a large percentage of total service load, the 0.45 f'_c limit may control. On the other hand, when a large portion of the total service load consists of a transient or temporary service live load, the increased stress limit of 18.4.2(b) [0.6 f'_c] may apply.
>
> The compression limit of 0.45 f'_c for prestress plus sustained loads will continue to control the long-term behavior of prestressed members."

2.6 CRITICAL MECHANICAL PROPERTIES OF HSC FOR DESIGN

HSC and HPC are two terms that are sometimes used interchangeably. For HPC, the aspect of most interest is not the strength, which may or may not be above normal, but the ease of placement, long-term mechanical properties, early-age strength, toughness, or service life in service environments. Therefore, HPC involves more attributes than high strength. The special performance and uniformity requirements cannot always be achieved by using only conventional materials with normal mixing, placing, and curing practices.

On the other hand, HSC has other physical properties that also make it a highly desirable material. Its modulus of rupture (MOR), drying shrinkage and creep, porosity, permeability, durability, corrosion resistance, thermal properties, and bond to steel are properties that tend to be superior to those for NSC (Farny and Panarese 1994). HSC is achieved through proper selection of materials, mixture proportioning, mixing, placing, curing, quality control, and testing. In this study the definition of HSC is concrete with specified compressive strengths for design of 6000 psi or greater, made without using exotic materials or techniques (ACI 363 1997).

Code changes need to address differences found for HSC, and it is important to understand how HSC behaves differently from NSC to ensure a conservative design when using codes developed for NSC. Relevant information for the MOR (f_r) of HSC is provided below because this is an important property related to the allowable tensile stress.

Adelman and Cousins (1990) reported material properties for HSC. A MOR of 1360 psi (28 days) for a concrete strength of 11460 psi was obtained in this study, which would yield the relationship $12.7\sqrt{f'_c}$. This value is consistent with values reported in other literature for HSC (ACI 363, 1997), as well as with the equation proposed by Carrasquillo et al. (1981), $f_r = 11.7\sqrt{f'_c}$, for concrete strengths ranging from 3000 to 12000 psi. The current code (AASHTO 2002 a,b) equation, estimating $f_r = 7.5\sqrt{f'_c}$ (which was developed for NSC), yields a MOR significantly lower than the 1360 psi found in the study by Adelman and Cousins (1990).

Phase 1 of this project (Hueste et al. 2003b) analyzed current prediction formulas that relate mechanical properties to the compressive strength to determine whether they can be used with sufficient accuracy for HSC produced by Texas precasters. Phase 1 evaluated prediction relationships for the modulus of elasticity, splitting tensile strength, and MOR. HSC samples for three specified concrete strength ranges (f'_c = 6000, 8000 and 10000 psi) were collected from three Texas precasters. In this study, the best-fit prediction formula for the MOR is $f_r = 10\sqrt{f'_c}$ for samples cured in the laboratory after curing in the field for the first day. Note that this equation does not provide a lower bound. Phase 3 of this project (Hueste et al. 2003c) assessed the impact of different curing conditions on the compressive strength and MOR of HSC and found that different field curing conditions can lead to a significant reduction in the MOR. It should be noted that Phase 3 was ongoing during the study documented within this report, and therefore the Phase 3 findings were not directly implemented in the evaluation of different tensile stress limits discussed later in this report. However, an evaluation of the final recommendations from Phase 3 are provided by Hueste et al. (2003c).

Nilson (1985) indicated that due to curing conditions in the laboratory and in situ, the actual MOR of a structural member would be less than that obtained in the laboratory. Nilson recommends the use of the $7.5\sqrt{f'_c}$ as a conservative value for MOR. However, it should be noted that this equation was based on normal concrete strengths.

2.7 CONCRETE STRENGTHS AT TRANSFER

Stresses at release and daily production requirements result in the need for a high early concrete strength at transfer of the strand prestress to the girder. Moreover, research shows that longer span lengths can be achieved using HSC. However, because the additional capacity in some cases also requires higher initial concrete strengths at transfer, the allowable tensile stresses at release may control designs.

Dolan et al. (1993) focused on the increased strength due to aging in HSC through examination of the historical strength gain and determined if additional final design strength is available. They reported that when additional span capacity comes at the expense of additional prestressing strands, the additional prestress results in a larger initial tensile stress, and consequently, larger initial concrete strengths during transfer are required. However, there was not an established limit for the allowable concrete tensile stress at transfer in their study. They concluded that many prestressed concrete girders display an actual concrete strength in excess of the specified strength due to the strength gain after release that brings the 28-day strength to be well above the specified 28-day requirement. The post-28 day strength gain depends on the mixture proportions. Precasters often use mixture proportions that gain strength after release. Mixtures that gain most of their strength before release and have much less strength gain between 18 hours and 28 days are also possible. In the case of HSC, early strength gain is typically necessary.

3 FLEXURAL DESIGN SPECIFICATIONS FOR PRESTRESSED CONCRETE BRIDGE GIRDERS

3.1 GENERAL

This section provides a review of current design provisions for prestressed concrete bridge girders for existing specifications with a focus on the AASHTO LRFD Specifications. Equations for prestress losses are presented. Significant changes in flexural design provisions in the AASHTO LRFD Specifications versus the AASHTO Standard Specifications are noted. Finally, relevant TxDOT documents are also reviewed.

3.2 AASHTO STANDARD AND LRFD SPECIFICATIONS

3.2.1 Philosophy

As of 2002, AASHTO had issued two design specifications for highway bridges: the AASHTO Standard Specifications for Highway Bridges, 16th Edition and 2002 Interim Specifications (AASHTO 2002a) and the AASHTO LRFD Bridge Design Specifications, 2nd Edition and 2002 Interim Revisions (AASHTO 2002b). In 2003, the Standard Specifications for Highway Bridges, 17th Edition, was released (AASHTO 2003). This study references AASHTO (2002 a,b).

The AASHTO Standard Specifications use the ASD and the LFD philosophies. Whereas the AASHTO LRFD Specifications, referred to as load and resistance factor design, are written in a probability-based limit state format. For prestressed concrete design, both specifications also include service load design criteria.

ASD was developed on the premise that safety can be established based primarily on experience and judgment. Allowable stresses are indicators of resistance and are compared with the resultant stresses from the elastic analysis of a structural member under design loads. Design events are specified through the use of load combinations, but they do not recognize that some combinations of loading are less likely to occur than others.

In LFD, a preliminary effort was made to recognize the variability of some loads. This variability is considered by using a different multiplier for dead, live, and other loads. Thus, loads are multiplied by factors and then added to produce load combinations. However, LFD does not take into account the statistical variability of design parameters.

In contrast, LRFD is written in a probability-based limit state format where safety is provided through the selection of conservative load and resistance factors. Load and resistance factors are determined for each limit state considered, and safety is measured in terms of the target reliability index (Nowak and Collins 2000). Unlike the AASHTO Standard Specifications, the LRFD provisions allow for a more constant and uniform safety level for various groups of bridges and construction types.

Numerous engineering disciplines use design based on probability, but its application to bridge engineering has been relatively small. As an example, conventional code calibration methods assume that the maximum factored design load effect governs designs, and then load and resistance factors are determined for the ultimate limit state (Tabsh 1992). However, designs for prestressed concrete and composite steel girder bridges are still governed by the allowable stress requirements (stress limits). Therefore, in these cases, both the SLS and the ULS should be considered.

3.2.2 Design Criteria for Prestressed Concrete Bridge Girders

This study focuses on the elastic flexural behavior of simply supported beams. Shear stresses are generally not a controlling factor in limiting maximum span lengths; therefore, the shear limit state was not considered in this study.

3.2.2.1 AASHTO Standard Specifications

In the AASHTO Standard Specifications, prestressed concrete bridge girders are designed to satisfy ASD and LFD. To satisfy ASD, the prestressed concrete bridge girders must satisfy allowable initial and final stresses at service load conditions. To satisfy LFD, the ultimate flexural capacity of the section is checked. The AASHTO Standard Specifications call for the structure to be able to withstand different load combinations for every group corresponding to the ASD and LFD; however, only a few groups govern the designs. The general design equation, the groups, and the load combinations for ASD and LFD are given by the equation shown below.

$$\phi R_n = 1.0\, R_n \geq \text{Group } (N) = \gamma\, \Sigma\, [\, \beta_i L_i\,] \tag{3.1}$$

where:

- ϕ is the resistance factor (=1.0 for flexural design of factory-produced precast prestressed members),
- R_n is the nominal resistance,
- N is the group number (ASD and LFD = I),
- γ is the load factor (1.0 for service and varies for LFD design based on grouping, I = 1.3),
- β_i is a coefficient that varies with the type of load and depends on the load group and design method (ASD = 1.0 for dead, live and impact loads; LFD = 1.0 for dead load and 1.67 for live and impact loads), and
- L_i is the force effect (D = dead load, L = live load, I = live load impact).

The resistance factor ϕ, the group loading coefficients β, and the load factors γ are available in the AASHTO Standard Specifications. Thus, applying Equation 3.1, and the corresponding coefficients, the design equations for ASD (service) and LFD (strength) are as follows.

Service (ASD):

$$\text{Group } (1) = 1.0(D) + 1.0(L+I) \tag{3.2}$$

Strength (LFD):

$$\text{Group (1)} = 1.3[1.0(D) + 1.67(L+I)] \tag{3.3}$$

3.2.2.2 AASHTO LRFD Specifications

In the AASHTO LRFD Specifications, bridges are designed for specified limit states to achieve the objectives of safety, serviceability, and constructability with due regard to issues of inspectablity, economy, and aesthetics. Because safety is the most important aspect of design, it is addressed in this section, while the other aspects are secondary but also important. In safety, four basic limit states must be satisfied: (1) service, (2) fatigue and fracture, (3) strength, and (4) extreme event limit states, where all limit states are considered of equal importance. However, like the AASHTO Standard Specifications, prestressed concrete bridge girders are designed in LRFD to satisfy the service limit states; the strength limit state is an additional, but important, limit state that must also be satisfied.

In this study, actions to be considered in the service limit states are the concrete stresses. The service limit states are based in part on experience related to provisions that cannot always be derived solely from strength or statistical considerations. Service limit states for prestressed concrete bridge members include Service I and Service III limit states. Service I is a load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Compression in prestressed concrete components is investigated using this load combination. Service III is a load combination relating only to tensile stresses in prestressed concrete structures with the objective of crack control (AASHTO 2002b).

The Strength I limit state is the base load combination relating to the normal vehicular use of the bridge without wind (AASHTO 2002b). Thus, regardless of the type of analysis used, Equation 3.4 has to be satisfied for all specified force effects and combinations as specified for each limit state.

$$\phi R_n \geq Q = \Sigma [\, n_i \, \gamma_i \, Q_i \,] \tag{3.4}$$

where:

ϕ is the resistance factor (=1.0 for both the service limit states and the flexural strength limit state),

R_n is the nominal flexural resistance,

γ_i is the statistical load factor applied to the force effects (Service I: = 1.0 for the dead and live loads; Service III: = 1.0 for the dead and 0.8 for the live loads; Strength I: = 1.25 for the dead and 1.75 for the live loads),

Q_i is the force effect (DC = dead load of structural components and nonstructural attachments, LL = live load, IM = live load impact), and

n_i is the load modification factor ($n_i = n_D\, n_R\, n_I \geq 0.95$)

where n_D is the ductility factor (for this study, for Strength I, n_D = 1.0 for conventional designs and details that comply with the specifications; for Service I and Service III, n_D = 1.0), n_R is the redundancy factor (for this study, n_R = 1.0 for conventional levels of redundancy for Strength I; Service I and Service III, n_R = 1.0), and n_I is the operational importance factor (for this study, n_I = 1.0 for typical bridges for Strength I; Service I and Service III, n_I = 1.0). Therefore, for this study, n_I was taken as 1.0 for the Service I, Service III, and Strength I flexural limit states.

The conversion to a probability-based LRFD methodology could be thought of as a mechanism to select the load and resistance factors more systematically and rationally than was done with the information available when ASD and LFD designs were introduced. Moreover, comparison of Equations 3.1 and 3.4 shows that LRFD requires consideration of ductility, redundancy, and operational importance. These are important issues, which affect the margin of safety of bridges, that are quantified in the AASHTO LRFD Specifications.

Applying the corresponding coefficients in Equation 3.4 the load equations for Service I, Service III, and Strength I are as follows.

Service I (compression):

$$Q = 1.0(DC) + 1.0(LL{+}IM) \tag{3.5}$$

Service III (tension):

$$Q = 1.0(DC) + 0.8(LL+IM) \quad (3.6)$$

Strength I (ultimate flexural strength):

$$Q = 1.0[1.25DC + 1.75(LL+IM)] \quad (3.7)$$

The statistical significance of the 0.8 factor on live load for the Service III limit state is that the design event is expected to occur about once a year for bridges with two traffic lanes, less often for bridges with more than two traffic lanes, and about once a day for bridges with a single traffic lane. Service III is used to investigate tensile stresses in prestressed concrete components (AASHTO 2002b).

Sections 5.3.3 and 5.3.5, respectively, provide equations used to satisfy serviceability and the ultimate capacity in more detail. Allowable stresses for concrete specified by both the AASHTO Standard and LRFD Specifications are shown in Table 3.1. These stress limits for concrete are used to control flexural stresses in extreme fibers at any section along the member. Stress limits for prestressing tendons for both specifications are shown in Table 3.2.

Table 3.1. Allowable Stresses Specified by the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b).

Type of Stress		Allowable Stresses (psi)
Initial Stage: Immediately after Transfer (After Initial Loss in the Prestressing Force)	Tension	200 or $3\sqrt{f'_{ci}}$ *
	Compression	$0.6 f'_{ci}$
Intermediate State: After Cast-in-Place Concrete Slab Hardens. Only Sustained Loads. (After Final Loss of Prestressing Forces)	Compression	Standard: $0.4 f'_c$ LRFD: $0.45 f'_c$
Final Stage: Total Dead and Live Loads (After Final Loss of Prestressing Forces)	Tension	$6\sqrt{f'_c}$
	Compression	$0.6 f'_c$
Additional Check of the Compressive Stress at the Final Stage	Compression	$0.4 f'_c$

Note:
* When the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete. The maximum tensile stress shall not exceed $7.5\sqrt{f'_{ci}}$ for the Standard Specifications and $6.96\sqrt{f'_{ci}}$ for the LRFD Specifications ($0.22\sqrt{f'_{ci}}$ in ksi).

Table 3.2. Stress Limits for Low Relaxation Prestressing Tendons Specified by the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b).

Condition	Stress Limit
Immediately Prior to Transfer (After Initial Loss in the Prestressing Force)	0.75 f_{pu}
At Service Limit States (After Final Loss of Prestressing Forces)	0.80 f_{py}

Notes:
f_{pu} = Specified tensile strength of prestressing steel (270 ksi low relaxation strand)
f_{py} = Yield strength of prestressing steel

3.3 PRESTRESS LOSSES

3.3.1 General

This section provides the prestress loss equations used for pretensioned members for both the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b). The prestressing force initially applied to a concrete member decreases with time due to several sources of losses. The reduction of the prestressing force is grouped into two categories: immediate losses and time-dependent loss. For pretensioned girders, the immediate losses include elastic shortening and steel relaxation; the time-dependent losses include concrete creep, concrete shrinkage, and steel relaxation.

The estimation of the magnitude of prestress losses is not exact because they depend on several factors. Moreover, the empirical methods vary with the different codes or practice. Both the AASHTO Standard and LRFD Specifications provide two methods for estimating prestressing losses: detailed computation and lump sum estimate. The method to be applied depends on the accuracy required. In common practice, the detailed method is not necessary and a lump sum estimate may be sufficient because a high degree of refinement is not desirable or warranted. However, for cases where more accuracy is required, the detailed method is more suitable because the losses are computed separately. Additional accuracy may be achieved by using the time-step method, which accounts for the interdependence of time-dependent losses using discrete time intervals. The detailed method, described by the following code equations, was used in this study.

3.3.3 AASHTO Standard Specifications

The main equations used in this study for calculation of the detailed prestressed losses for designs following the AASHTO Standard Specifications are described below.

3.3.3.1 Immediate Losses

3.3.3.1.1 Elastic Shortening

The prestress loss due to elastic shortening for pretensioned members is computed as follows,

$$E_s = \frac{E_{ps}}{E_{ci}} f_{cir} \tag{3.8}$$

where E_s is the loss due to elastic shortening, E_{ps} is the modulus of elasticity of the prestressing steel strands, E_{ci} is the modulus of elasticity of concrete at release, and f_{cir} is the concrete stress at the centroid of the prestressing steel due to the prestressing force and self-weight of the beam immediately after transfer. The expression to determine f_{cir} is shown in the following equation,

$$f_{cir} = \frac{-P_i}{A}\left[1 + \frac{e^2}{r^2}\right] + \frac{M_D\, e}{I} \tag{3.9}$$

where P_i is the initial prestressing force, A is the cross-sectional area of the precast section, e is the eccentricity of tendons from the center of gravity of the concrete section (cgc), r is the radius of gyration, M_D is the moment due to self-weight of the precast section (dead load), and I is the moment of inertia of the precast section.

3.3.3.1.2 Steel Relaxation

An approximation was made in the computation of prestress loss due to steel relaxation at transfer (immediate loss) in order to match the TxDOT design procedure. TxDOT considers approximately one-half of the total amount of the steel relaxation loss for loss at transfer (immediate loss) and the same amount for loss after transfer (time-dependent losses). The equation for the total loss due to steel relaxation for pretensioned members using low relaxation strands is as follows,

$$CR_s = 5000 - 0.1E_s - 0.05\left(S_H + CR_C\right) \tag{3.10}$$

where CR_s is the loss due to relaxation of the prestressing strands (psi), E_s is the loss due to elastic shortening (psi), S_H is the loss due to concrete shrinkage (psi), and CR_c is the loss due to creep of concrete (psi). The values for E_s, CR_C, and S_H are given by Equations 3.8, 3.11, and 3.13, respectively.

3.3.3.2 Time-Dependent Losses

3.3.3.2.1 Concrete Creep

The equation for the prestress loss due to concrete creep for pretensioned members is as follows,

$$CR_C = 12\ f_{cir} - 7f_{cds} \tag{3.11}$$

where CR_C is the loss due to concrete creep, f_{cir} is the concrete stress at the centroid of the prestressing steel due to the prestressing force and the self-weight of the beam immediately after transfer (Equation 3.9), and f_{cds} is the concrete stress at the centroid of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied. The value of f_{cds} is found using the following equation,

$$f_{cds} = \frac{M_{SD}\,e}{I} + \frac{M_{CSD}}{I_C}\left(e + c_{cb} - c_b\right) \tag{3.12}$$

where M_{SD} is the moment due to the self-weight of the slab (superimposed dead load), M_{CSD} is the moment due to the self-weight of the rail (composite superimposed dead load), e is the eccentricity of tendons from the concrete section center of gravity (cgc), I is the moment of inertia of the precast section, I_C is the moment of inertia of the precast section and slab composite section, c_b is the distance from the cgc to the extreme bottom fiber of the precast section, and c_{cb} is the distance from the cgc to the extreme bottom fiber of the composite section.

3.3.3.2.2 Concrete Shrinkage

The expression for the prestress loss due to concrete shrinkage for pretensioned members is as follows,

$$S_H = 17000 - 150\ RH \tag{3.13}$$

where S_H is the loss due to concrete shrinkage (psi) and RH is the mean annual ambient relative humidity in percent.

3.3.3.2.3 Steel Relaxation

The equation for the total prestress loss due to steel relaxation for pretensioned members using low relaxation strands is given in Equation 3.10. One-half of this total amount is considered for the losses after transfer (time-dependent losses).

3.3.4 AASHTO LRFD Specifications

The main equations used in this study for calculation of the detailed prestressed losses for designs following the AASHTO LRFD Specifications are described below.

3.3.4.1 Immediate Losses

3.3.4.1.1 Elastic Shortening

The equation (3.14) for the prestress loss due to elastic shortening (Δf_{pES}) for pretensioned members is the same as that for the AASHTO Standard Specifications shown in Equation 3.8.

$$\Delta f_{pES} = \frac{E_{ps}}{E_{ci}}\ f_{cir} \tag{3.14}$$

3.3.4.1.2 Steel Relaxation

The equation for the loss due to steel relaxation at transfer for pretensioned members using low relaxation strands is as follows,

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj} \qquad (3.15)$$

where Δf_{pR1} is the loss due to relaxation at transfer, f_{pj} is the initial stress in the tendon after stressing is complete, f_{py} is the specified yield strength of prestressing steel, and t is the estimated time from stressing to transfer (days).

3.3.4.2 Time-Dependent Losses

3.3.4.2.1 Concrete Creep

The equation for the prestress loss due to concrete creep (Δf_{pCR}) for pretensioned members is the same as that for the AASHTO Standard Specifications shown in Equation 3.11.

$$\Delta f_{pCR} = 12 f_{cir} - 7 f_{cds} \geq 0 \qquad (3.16)$$

3.3.4.2.2 Concrete Shrinkage

The equation for the prestress loss due to concrete shrinkage (Δf_{pSR}) for pretensioned members is the same as that for the AASHTO Standard Specifications shown in Equation 3.13. In this case Δf_{pSR} is in ksi.

$$\Delta f_{pSR} = 17.0 - 0.15 RH \qquad (3.17)$$

3.3.4.2.3 Steel Relaxation

Losses due to relaxation of prestressing strands with low relaxation properties for pretensioned members is 30 percent of the prestress losses due to the steel relaxation of prestressing steels with stress-relieved strands (Δf_{pR2}) given in Equation 3.18. In this case Δf_{pR2}, Δf_{pES}, Δf_{pSR}, *and* Δf_{pCR} are in ksi.

$$\Delta f_{pR2} = 20.0 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}) \tag{3.18}$$

3.4 SIGNIFICANT CHANGES IN THE AASHTO LRFD SPECIFICATIONS

3.4.1 General

Table 3.3 shows some important differences between the flexural design provisions for prestressed concrete members when comparing the AASHTO Standard and LRFD Specifications. The table presents these differences in the order they are provided in these documents. The significant changes are the introduction of the new live load model, a new expression for the impact loads, and new expressions for the live load distribution factor, as well as changes in the limit states. These changes are discussed below.

3.4.2 Live Load Model

The live load is an important factor for bridge design. There is a significant change in the live load model specified in the LRFD Specifications. The new HL93 live load model is a superposition of the HS20 truck and the uniform lane load. In contrast, both of these components are considered separately, with the maximum effect used for design in the AASHTO Standard Specifications. The HL93 live load model is intended to be representative of the truck population and is considered to provide a consistent safety margin for a wide spectrum of spans. Table A.1 (Appendix A) shows a comparison of distribution factors and design live load moments for U54 Beams. This table also provides a comparison of LRFD and Standard Specifications live load moments "per lane" with the objective to compare only the effect of the new live load model. Percentage increases between 48 and 70 percent (varying with the span lengths but constant for a given span) were found for the LRFD Specifications. These percentages show the significance of the new LRFD live load demands in terms of the lane moments. However, refinements in the expressions used to compute the live load distribution factor help to reduce the LRFD design live load moment per girder as shown in the next section.

Table 3.3. Important Differences between the Flexural Design Provisions for Prestressed Concrete Bridge Girders in the AASHTO Standard and LRFD Specifications.

Description	**AASHTO Standard Specifications**	**AASHTO LRFD Specifications**
Live Load	a) Standard HS20 Truck b) HS20 Lane Loading c) Tandem: Military Loading Whichever produces maximum stresses	a) HS20 Truck and Lane Loading (HL93) b) Tandem and HS20 Lane Loading Whichever produces maximum stresses
Dynamic Load (I) L = Span length	$I = \dfrac{50}{L+125} \leq 30\%$	0.33
Lateral Distribution L = Span length; S_{int} = Girder Spacing for interior beams; d = Girder depth; t = Slab depth; E_{beam} = Modulus of elasticity of the girder; E_{slab} = Modulus of elasticity of the slab; A = Area of girder; I = Moment of inertia; e_g = Dist. between the centers of gravity of the precast beam and cast-in-place slab	Any Internal beam: $S_{int}/11$ per truck/lane with a minimum value of 0.9	<u>*U54 Beam:*</u> Int. Beams: $\left(\dfrac{S_{int}}{6.3}\right)^{0.6}\left(\dfrac{S_{int}d}{12L^2}\right)^{0.125}$ 6.0 ft. $\leq S_{int} \leq$ 11.5 ft., 20 ft. $\leq$ L $\leq$ 140 ft. 8 in. $\leq$ d $\leq$ 65 in., $N_{beams} \geq 3$ For $S_{int} \geq$ 11.5 ft., use the lever rule <u>*Type IV Beam:*</u> Int. Beams: $0.075 + \left(\dfrac{S_{int}}{9.5}\right)^{0.6}\left(\dfrac{S_{int}}{L}\right)^{0.2}\left(\dfrac{K_g}{12Lt^3}\right)^{0.1}$ $K_g = n\left(I + Ae_g^2\right)$; $n = E_{beam}/E_{slab}$ 3.5 ft. $\leq S_{int} \leq$ 16 ft., 20 ft. $\leq L \leq$ 240 ft. $N_{beams} \geq 4$ For $N_{beams} = 3$, use the lever rule
Limit States, Load Factors, and Combinations	<u>*Service:*</u> $Q = 1.0\ (D) + 1.0\ (L+I)$ <u>*Strength:*</u> $Q = 1.3\ [1.0\ (D) + 1.67\ (L+I)]$	<u>*Service I:*</u> $Q = 1.0\ (D+W) + 1.0\ (L+I)$ <u>*Service III:*</u> $Q = 1.0\ (D+W) + 0.8\ (L+I)$ <u>*Strength I:*</u> $Q = 1.0\ [1.25\ (D) + 1.75\ (L+I)]$
Resistance Factors (Prestressed Concrete)	$\phi Rn \geq \gamma \Sigma [\beta_i L_i]$ Resistance factor ϕ=1.0	$\phi Rn \geq \Sigma [n_i \gamma_i L_i]$ Resistance factor ϕ: 1) Strength limit states: Flexure and tension ϕ=1.0 2) Non-strength limit states: ϕ=1.0
Losses Variables are defined in Sections 3.3.1 and 3.3.2	Δf_{pR1}= loss due to relaxation at transfer Δf_{pR2}=loss due to relaxation after transfer Δf_{pR} = total loss due to relaxation For low relaxation strand: $\Delta f_{pR} = 5.0 - 0.10\Delta f_{pES} - 0.05(\Delta f_{pSR} + \Delta f_{pCR})$ $\Delta f_{pR1} = \Delta f_{pR2} = \Delta f_{pR} / 2$	Δf_{pR1} = loss due to relaxation at transfer Δf_{pR2}= loss due to relaxation after transfer For low relaxation strand: $\Delta f_{pR1} = \dfrac{\log(24.0t)}{40.0}\left[\dfrac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$ For pretensioning with stress-relieved stands: $\Delta f_{pR2} = 20 - 0.4\ \Delta f_{pES} - 0.2\ (\Delta f_{pSR} + \Delta f_{pCR})$*

Note:

* For pretensioning with low relaxation strands: Δf_{pR2} = 30% of equation

3.4.3 Live Load Distribution Factors

The AASHTO LRFD Specifications provide new approximate methods of analysis for the distribution factors (DFs) for typical materials and bridge sections based on an accurate analysis and a statistical approach (AASHTO 2002b). However, these DFs are still considered to be conservative since they are based on analyses for typical bridges.

3.4.3.1 Texas U54 Beams

In this study, the DFs for Texas U54 Beams were computed according to both the AASHTO Standard and LRFD Specifications (see Table 3.3). For the LRFD Specifications, the DFs for the U54 Beams were determined using expressions for the typical cross-section "c", which is referred to as "cast-in-place concrete slab on open precast concrete boxes." However, for the case of U54 Beams designed under the LRFD Specifications, the DFs are significantly smaller than those obtained through the use of the simplified DF expression *S*/11, where *S* is the girder spacing, provided in the AASHTO Standard Specifications.

Table A.1 (Appendix A) shows DFs for U54 Beams computed using both the Standard and LRFD Specifications. The calculations were performed for spans ranging from 90 to 140 ft., and for each span five girder spacings were considered. For girder spacings less than 11.5 ft., the DFs using the LRFD Specifications are 24 to 39 percent larger than the Standard Specifications DFs, varying with the span lengths and girder spacings. However, the DFs for the wider girder spacings are almost the same for both specifications, with an increase of 4.6 percent for the LRFD Specifications. Where the lever rule was applied for LRFD designs (for wider girder spacings), the resulting DFs for the Standard and LRFD designs are almost the same.

3.4.3.2 Type IV Beams

The DFs for the Type IV beams were determined using the equation provided in the LRFD Specifications shown in Table 3.3. For the AASHTO Standard Specifications the simplified expression $S/11$, where S is the girder spacing, provided in the Standard Specifications, was used.

Table A.2 (Appendix A) shows DFs for AASHTO Type IV beams, computed using both the Standard and LRFD Specifications. The calculations were performed for spans ranging from 90 to 140 ft., and for each span five girder spacings were considered. The DFs using the Standard Specifications are larger (from 2 to 17 percent larger) for wider girder spacings (8.3 and 7 ft. for all spans and 5.75 ft. for spans larger than 110 ft.). For shorter spans and smaller girder spacings, the DFs from the LRFD Specifications are larger (from 1 to 18 percent larger). These results show the same trends as those reported for the calibration of the AASHTO LRFD Specifications (Nowak 1999).

3.4.4 Design Live Load Moments

As noted earlier, the unfactored live load moments per lane were between 48 and 70 percent larger for the LRFD Specifications. This increase varied with the span length but was constant for each span considered.

3.4.4.1 U54 Beams

Table A.1 (Appendix A) shows the live load moment per U54 beam after applying the appropriate DFs for both the Standard and LRFD Specifications. The resulting live load moments per beam are between 9 and 81 percent larger for the LRFD Specifications (varying greatly with girder spacings and span lengths). However, when girder spacings wider than 11.5 ft. are not considered in the comparison, the range is reduced to a 9 to 29 percent increase for the LRFD Specifications. For girder spacings wider than 11.5 ft. the lever rule was applied for LRFD designs and the resulting DFs for the Standard and LRFD designs are almost the same. This trend shows that the LRFD DFs for U54 Beams greatly reduce the live moments per beam, relative to the $S/11$ expression.

Researchers made an evaluation of the use of the traditional $S/11$ DF expression from the Standard Specifications in conjunction with the live load model in the LRFD Specifications. Results in Table A.1 show that the resulting live load moments would be significantly larger (from 60 to 90 percent) than those from the Standard Specifications. In comparison, the use of the DF based on the LRFD Specifications led to a much smaller increase in the live load moment per U54 beam (9 to 29 percent) over the Standard Specifications. Therefore, the advantage of using the DF expression from the LRFD Specifications is clear and this factor was applied for the U54 Beams in this study.

3.4.4.2 Type IV Beams

Table A.2 (Appendix A) shows DFs and live moments per lane and per beam for AASHTO Type IV beams, computed using both the Standard and LRFD Specifications. The calculations were performed for spans ranging from 90 to 140 ft., and for each span five girder spacings were considered. The DFs using the Standard Specifications are from 2 to 17 percent larger for wider girder spacings (8.3 and 7 ft. for all spans and 5.75 ft. for spans larger than 110 ft.). For shorter spans and shorter girder spacing, the DFs using the LRFD Specifications are from 1 to 18 percent larger. However, the resulting live moment per beam shows percentage increases from 49 to 101 percent for LRFD designs, indicating that the effect of the new live load model on the LRFD live load moments per beam is significant. Results show that the Type IV beam DFs do not vary as much for the two specifications as was observed for the U54 Beams.

3.4.5 Limit States

Three limit states are considered for LRFD prestressed concrete bridge girder designs: Service I to check compressive stresses, Service III to check tensile stresses, and Strength I to check the ultimate flexural moment. The Service III limit state is an addition to the traditional design limit states included in the Standard Specifications, where allowable stresses at initial and final load conditions (service) must be satisfied and the ultimate flexural capacity is also checked. In the Service III limit state, a 0.8 factor is applied to the live load, where the tension under live load is being investigated with the objective of crack control. This factor was introduced in the LRFD Specifications to help to compensate for the additional live load effect on the tensile stresses resulting from the use of the HL93 live load model. The statistical significance of the 0.8 factor on live load for the Service III limit state is that the design event is expected to occur about once a year for bridges with two traffic lanes, less often for bridges with more than two traffic lanes, and about once a day for bridges with a single traffic lane.

3.5 TxDOT DESIGN GUIDELINES AND SOFTWARE

TxDOT has in-house design guidelines and software that were referred to during this study to ensure consistency with current TxDOT practices. This section describes the relevant documents.

TxDOT has issued its own Bridge Design Manual (TxDOT 2001a) and a Preliminary Design Guide for U-Beam Bridges (TxDOT 2001b). These documents are intended to promote consistency in design and details. Supplemental design criteria for prestressed concrete U54 Beams, such as geometric properties, debonded lengths, composite and non-composite dead load requirements, maximum girder spacings, and maximum recommended span lengths, were taken from the TxDOT Preliminary Design Guide for U-Beam Bridges. Supplemental design criteria for the prestressed concrete Type IV beams, such as geometric properties, hold-down points, harping, and maximum girder spacings, maximum recommended span lengths, were taken from the TxDOT Bridge Design Manual. Some design parameters for the Type IV beam designs, such as the composite and non-composite dead load requirements, were considered to be the same as those for the U54 beam designs in order to compare the designs.

Finally, TxDOT uses the PSTRS14 (prestressed concrete bridge girder design program) (TxDOT 1980) is used by TxDOT to design and analyze standard I, standard box, and non-standard beams with either draped or straight (partially debonded) strands. This program has been updated for design according to the AASHTO Standard Specifications for Highway Bridges, 16th Edition (AASHTO 1996). Standard double T-beams and standard U-beams may also be designed and analyzed as non-standard beams.

4 SURVEY OF CURRENT PRACTICE

4.1 INTRODUCTION

A survey entitled "Current Practice for Design of High Strength Concrete Prestressed Members" was developed and distributed to all 52 state departments of transportation (DOTs) and to six other organizations involved in the design of bridge structures. The objective of this survey was to gather information and document critical aspects of current practice for the design of HSC prestressed bridge girders. Responses from 41 state DOTs and two private organizations were collected, giving a 74 percent response rate. A copy of the complete questionnaire is provided in Appendix B.

The questionnaire consists of two parts. Part I: Current Design Practice for HSC Prestressed Bridge Members contains 11 questions related to current specifications, additional documents and references, construction using HSC, typical range of specified concrete strengths, concrete strengths at transfer and at service, concerns related to the use of HSC, and adjustments of the design specifications for HSC. Part II: Description of Typical Bridges using HSC Prestressed Bridge Members queries for information on the span lengths and concrete strengths for each type of bridge in which HSC has been used by the respondents. In the following sections, tables summarizing the responses are presented.

Table 4.1 provides a list of respondents to the questionnaire. Of the 41 state DOTs that provided a response, only one did not give permission to identify their organization when reporting their response. This DOT is identified as "Undisclosed DOT." The two structural engineering firms listed are located in the state of Texas and were identified by TxDOT to be recipients of the survey.

Table 4.1. List of Respondents.

Department of Transportation	Department of Transportation
Alabama	New Jersey
Alaska	New Mexico
Arkansas	New York
California	North Carolina
Colorado	North Dakota
Connecticut	Ohio
Florida	Oklahoma
Georgia	Pennsylvania
Hawaii	Rhode Island
Idaho	South Carolina
Illinois	South Dakota
Iowa	Tennessee
Kansas	Texas – Austin
Kentucky	Vermont
Louisiana	Virginia
Massachusetts	Washington
Michigan	Wisconsin
Minnesota	Undisclosed DOT
Mississippi	
Missouri	**Additional Respondents**
Montana	Texas – Houston
Nevada	Structural Engineering Associates
New Hampshire	Turner, Collie & Braden, Inc.

4.2 PART I: CURRENT DESIGN PRACTICE FOR HSC PRESTRESSED MEMBERS

4.2.1 Current Specifications

The first three questions of the survey address the current specifications in use for bridge design. The questions are as follows.

Q 1: Current specification used by your organization for bridge member design.

Q 2: If your organization is currently using the AASHTO LRFD Specifications, when were they implemented in your state (provide year)?

Q 3: If your organization plans to use the AASHTO LRFD Specifications in the future, when do you foresee their implementation in your state (provide year)?

Table 4.2 shows the responses related to current specifications in use for bridge design and the implementation of the AASHTO LRFD Specifications. The survey indicates that the changeover to the LRFD Specifications is still gradual. The AASHTO Standard Specifications for Highway Bridges, 16th Edition (AASHTO 1996), is still the most popular code for bridge design in current practice.

For the 41 DOTs involved in the survey, 78 percent are currently using the AASHTO Standard Specifications, 44 percent are using the AASHTO LRFD Specifications, and 22 percent are using both specifications. In most cases where the LRFD Specifications are used their implementation is partial, and most states plan complete implementation in the period of 2003 to 2007.

Table 4.2. Current Specifications.

Department of Transportation	Q 1		Q 2	Q 3
	Current Specification		LRFD is used	LRFD is not used
	LRFD	Standard	Date of Implementation	Date of Expected Implementation
Alabama		x (2000)		2007
Alaska	x (-)		1997 (partial)	2007
Arkansas		x (1996)		2007
California		x (2000)		2004
Colorado	x (current)		2000	
Connecticut	x (-)	x (-)	2000 (partial)	2004
Florida	x (1998)	x (1996)	1998	
Georgia		x (-)		2005
Hawaii	x (1998)		1996	
Idaho	x (2001)		2000	
Illinois		x (1996)		2007
Iowa	x (1998)	x (1996)	2000 (partial)	2003
Kansas	x (1998)		1999	
Kentucky		x (current)		2007
Louisiana	x (current)	x (1996)	2001(partial)	2005
Massachusetts	x (-)	x (1996)	1998 (partial)	2007
Michigan		x (1996)		2007
Minnesota	x (1998)	x (1996)	1998 (partial)	2002
Mississippi		x (1996)		2005
Missouri		x (1996)		2005
Montana	x (1998)	x (1996)	1994	
Nevada		x (1996)		2003
New Hampshire		x (1996)		2003
New Jersey	x (1996)		2000	

Table 4.2. Continued.

Department of Transportation	Q 1		Q 2	Q 3
	Current Specification		LRFD is used	LRFD is not used
	LRFD	Standard	Date of Implementation	Date of Expected Implementation
New Mexico		x (1996)		2007
New York		x (1996)		2005
North Carolina		x (1996)		2007
North Dakota		x (1996)		
Ohio		x (1996)		
Oklahoma	x (-)			
Pennsylvania	x (1998)		1997	
Rhode Island		x (1996)		2007
South Carolina	x (-)	x (-)	2000	
South Dakota		x (1996)		2007
Tennessee		x (1996)		2007
Texas – Austin	x (-)	x (-)	2002 (partial)	2007
Vermont		x (2001)		2003 – 2004
Virginia		x (1996)		2007
Washington	x (1998)		1995, 1998	
Wisconsin		x (2000)		2005
Undisclosed DOT		x (2002)		
Additional Respondents				
Texas – Houston		x (1996)		2005
Structural Engineering Associates		x (current)		2007
Turner, Collie & Braden, Inc.	x (-)	x (1996)	1999, 2000	2006

Specifically in the state of Texas, the LRFD Specifications are partially used. The Austin office reported that the LRFD Specifications have been partially used since 2002. The Houston District office reported that the LRFD Specifications have not yet been implemented by their group.

4.2.2 Additional Design Documents and References

Questions four and five of the survey request information on additional relevant design documents used by the respondents. In particular, these questions are as follows.

Q 4: Please list any other documents used by your organization for the design of prestressed concrete bridge girders.

Q 5: Please list any additional reference documents used by your organization for design of HSC members.

Table 4.3 shows additional design documents and references for respondents to this question. There are several documents other than the AASHTO Specifications that some organizations utilize in design. The survey shows that about one-third of the state DOTs use additional documents and references for the design of prestressed concrete bridge girders and HSC members. Among these documents and references are the PCI Bridge Design Manual (PCI 1997), some publications on HSC issued by the Portland Cement Association, bridge design manuals developed by individual state DOTs, software programs developed by state DOTs or software companies, and other reports and texts.

In Texas, the PCI Manual (1997) and the PSTRS14 Program Manual (TxDOT, 1980) are used as additional documents and references. TxDOT also uses the TX-Bridge Design Manual (TxDOT 2001a) and a Design Guide for U-Beam Bridges (TxDOT 2001b).

Table 4.3. Additional Documents and References.

Department of Transportation	Q 4 References for Prestressed Girder Design	Q 5 References for HSC Member Design
Alabama	PCI Manual	Texts
Alaska	Internal Procedures	
Arkansas	PCI / PCA	
California	CA-BD-Manual	PCI Manual
Colorado	CO-BD-Manual	
Illinois	PCI Manual	
Iowa	IA-BD-Manual	PCI Manual
Louisiana		Internal Research
Massachusetts	PCI Manual/Leap Software	
Michigan	PCI Manual	
Minnesota	PCI Manual	
Montana	PCI Manual	
New Hampshire	PCI Manual	
New Jersey	NJ-BD-Manual	
New York	NY-BD-Manual	PCI Manual
Ohio	PCA Publication	Texts
Pennsylvania	PA-BD-Manual	
Rhode Island		PCI Manual/ACI Code
South Carolina	PCI Manual	Leap Software
South Dakota	PCI Manual	Journals

Table 4.3. Continued.

Department of Transportation	Q 4	Q 5
	References for Prestressed Girder Design	**References for HSC Member Design**
Texas – Austin	PCI / PRSTRS14 Manuals	
Vermont	PCI Manual	
Virginia	ACI Code	
Washington	WS-BD-Manual	PCI-BD Manual
Additional Respondents		
Texas - Houston		
Structural Engineering Associates	TX-BD Manual	
Turner, Collie & Braden, Inc.	PRSTRS14 - Ubeam guide	

Note: BD Manual = Bridge Design Manual

4.2.3 HSC Prestressed Bridge Girder Precasters

Precasters names and locations were surveyed in question six, as follows.

Q 6: Please provide the names and locations of precasters that supply HSC prestressed girders for your bridge projects.

Table B.1 in Appendix B shows the name and locations of precasters that supply HSC prestressed girders for each DOT. A total of 35 state DOTs and two organizations responded to this question. The number of precasters that supply HSC prestressed girders used by each state DOT and their corresponding locations by states are also shown in Table B.1. The survey shows that a DOT may be served by one to as many as seven precasters. For example, five precasters supply Florida and Iowa, six precasters supply Massachusetts, and seven precasters serve North Carolina and Texas. Precasters serving a state DOT are not always located in the same state. For example, Florida has five precasters supplying HSC prestressed girders, of which two are located in other states (Mississippi and Georgia). Another example is Massachusetts where, from its six suppliers, four are located in four different states including Connecticut, Vermont, New York, and Nebraska.

Table B.2 in Appendix B provides a list of precasters and their different locations, as well as the different states each precaster serves. Some HSC precasters not only supply to their own state DOT but also other state DOTs. For example, Cretex is located in Minnesota but serves South Dakota. Morse Brothers, Inc. is located in Oregon but supplies Washington.

4.2.4 Prevalence of HSC Prestressed Bridge Girders

The number of HSC prestressed bridge girders constructed by state DOTs was surveyed in question 7, as follows.

Q 7: How many bridges does your organization typically construct each year ?
Of these, what percentage use HSC prestressed bridge girders (specified $f'_c > 6000$ psi)?

Table 4.4 shows the number of bridges that each state DOT typically constructs per year. A large variation, from 4 to 400, was observed. Of these numbers, the percentages of bridges constructed with HSC prestressed girders (from 0 to 100 percent) are also shown. Note that, in this study, the definition of HSC is concrete with specified compressive strengths for design of 6000 psi or greater, made without using exotic materials or techniques (ACI 363, 1997).

Table 4.4. Number of HSC Bridges Constructed (Q 7).

Department of Transportation	No. Bridges Constructed per Year	% HSC ($f'_c > 6$ ksi)
Alabama	50	50
Alaska	20	100
Arkansas	70	0
California	200	10
Colorado	48	40
Connecticut	22	0
Florida	60	90
Georgia	100	90
Hawaii	5	80
Idaho	10	85
Illinois	400	0
Iowa	30	90
Kansas	149	15
Kentucky	80	15
Louisiana	15	2
Massachusetts	20	0

Table 4.4. Continued.

Department of Transportation	**No. Bridges Constructed per Year**	**% HSC (f'_c > 6 ksi)**
Michigan	70	85
Minnesota	45	75
Mississippi	100	0
Missouri	250	1
Montana	18	60
Nevada	12	0
New Hampshire	30	10
New Jersey	31	1
New Mexico	10	25
New York	236	30
North Carolina	150	30
North Dakota	8	25
Ohio	150	30
Oklahoma	160	67
Pennsylvania	250	50
Rhode Island	4	100
South Carolina	50	5
South Dakota	12	10
Tennessee	80	60
Texas - Austin	360	18
Vermont	40	15
Virginia	125	30
Washington	30	80
Wisconsin	200	10
Undisclosed DOT	40	0
Additional Respondents		
Texas - Houston	50	75
Structural Engineering Associates	14	85
Turner, Collie & Braden, Inc.	45	85

In general, 68 percent of the responding DOTs use HSC prestressed girders for 0 to 50 percent of their total construction, 15 percent of the responding DOTs use HSC prestressed girders for 51 to 80 percent of their total construction, and 17 percent of the responding DOTs use HSC prestressed girders for 81 to 100 percent of their total construction. Among the DOTs with the highest rate of HSC prestressed bridge girders construction are the DOTs that also have a significant number of bridges constructed per year, such as Florida, Georgia, and Michigan. These states construct between 60 and 100 bridges per year. DOTs with the highest number of bridges constructed per year, such as Illinois (400), Texas–Austin (360), Missouri (250), Pennsylvania (250), New York (236), California (200), Wisconsin (200), and Ohio (150) have the lowest percentage of construction using HSC. For example, in Illinois, of 400 bridges built, no bridges were constructed using HSC.

In Texas, which constructs the second largest number of bridges per year (360), only 18 percent of the total construction utilized HSC prestressed girders, as reported by the Austin office. However, this does result in an important number of bridges using HSC prestressed girders in Texas. In TxDOT's Houston office, 75 percent of the total number of bridges constructed per year (50) would also be considered as an important number of bridges using HSC. Consequently, the use of HSC prestressed bridge girders in Texas is significant.

4.2.5 Specified Concrete Strength

Question eight of the survey focused on determining the range of the specified concrete strength for prestressed concrete bridge girders. The question is as follows.

> Q 8: Please provide the typical range of specified strength for prestressed concrete bridge girders used in current projects for your organization?

Table 4.5 shows typical ranges for specified concrete strength at transfer and service conditions for current projects. The required concrete strength at transfer ranges from 3500 to 9000 psi, while the required concrete strength at service ranges from 4000 to 12000 psi. Particularly, a typical range from 4000 to 6500 psi for f'_{ci} and from 5000 to 8500 psi for f'_c at service were reported by TxDOT's Austin office. A typical range from 4000 to 6200 psi for f'_{ci} and from 5000 to 8500 psi for f'_c at service was reported by TxDOT's Houston District office. The Austin office reported designs for longer spans that required f'_c up to 14000 psi (this strength was not reported as a typical value).

Table 4.5. Typical Range for Specified Concrete Strength for Prestressed Girders (Q 8).

Department of Transportation	**Range of Specified Concrete Strength**															
	***f'ci* at Transfer** (ksi)							***f'c* at Service** (ksi)								
	3	4	5	6	7	8	9	4	5	6	7	8	9	10	11	12
Alabama			x	x	x				x	x	x	x				
Alaska		x	x	x	x						x					
Arkansas		x							x							
California		x	x	x				x	x	x	x	x				
Colorado			x	x	x				x	x	x	x				
Connecticut		x							x	x						
Florida		x	x	x	x				x	x	x	x				
Georgia		x	x	x	x					x	x	x	x	X		
Hawaii		x	x	x						x	x					
Idaho		x	x						x	x						
Illinois			x							x						
Iowa		x	x	x	x				x	x	x	x	x			
Kansas		x	x						x	x						
Kentucky		x	x	x					x	x	x					
Louisiana		x	x						x	x						
Massachusetts		x	x	x						x	x	x				
Michigan	x	x	x	x					x	x	x					
Minnesota		x	x	x	x				x	x	x	x				
Mississippi		x	x						x	x						
Missouri				x	x						x	x	x	x		
Montana		x	x	x					x	x	x					
Nevada	x	x						x	x							
New Hampshire		x	x						x	x	x	x				
New Jersey		x							x	x						
New Mexico		x	x	x	x	x	x			x	x	x	x	x	x	x
New York			x	x	x						x	x	x	x		
North Carolina		x	x						x	x	x	x				
North Dakota		x	x	x					x	x	x					
Ohio			x								x					
Oklahoma																
Pennsylvania			x	x	x					x	x	x				
Rhode Island		x	x							x	x	x	x			
South Carolina	x	x	x						x	x	x	x				
South Dakota			x	x	x					x	x	x				
Tennessee		x	x	x	x	x			x	x	x	x	x	x		
Texas - Austin		x	x	x					x	x	x	x				
Vermont	x	x	x					x	x	x						
Virginia										x	x	x				
Washington					x	x							x			
Wisconsin			x							x	x	x				
Undisclosed DOT		x							x	x						
Additional Respondents																
Texas - Houston		x	x	x					x	x	x	x				
Structural Eng. Assoc.		x	x	x					x	x	x	x				
Turner, Collie & Braden		x	x	x	x				x	x	x	x	x			

The responses to the survey indicate that the most popular range for the concrete strength at transfer ranges from 4000 to 7000 psi, and from 5000 to 8500 psi for the concrete strength at service. About 7 percent of the DOTs utilize a higher concrete strength at transfer (8000 psi) for some cases, and 15 percent utilize a higher concrete strength at service (10000 psi) for some cases. Only 2 percent of the DOTs that responded utilize a concrete strength at service of 12000 psi, and 7 percent of DOTs utilize a lower concrete strength at service of 4000 psi. There is a prevalence for concrete strengths at service in the range of 6000 to 8000 psi (85 percent of total DOTs) indicating that HSC is widely used in current practice.

4.2.6 Impact of Required Transfer Strengths

The impact of high concrete strength requirements at transfer was surveyed in question nine, which is stated as follows.

Q 9: Please comment on whether the need to meet the required concrete compressive strength at transfer (f'_{ci}) in a short period of time has led to a practice where precasters use mix designs that give significantly a larger value of f'_c in service than specified.
If this practice has been observed by your organization, can you give any specific information as to how this overstrength varies as a function of specified f'_{ci} and f'_c values?

Table 4.6 identifies positive and negative responses to question nine as well as some specific information given by the state DOTs that responded to question nine.

Table 4.6. Specific Information for Required Transfer Strength (Q 9).

Department of Transportation	Yes	No	Specific Information
Alabama	x		Some use of high earlier strength additives are used if release is more than 6500 psi. Long-term strength gain is less when high early strength is attained.
Alaska	x		Recent job with f'_{ci}=7250 psi had 16 hr. break of 10000 psi. Same mix design later provided 6800 psi break.
Arkansas	x		This is being done but we do not observe lab tests for 28-day compressive strength.
California	x		Normally f'_c provided by precasters significantly exceeds f'_c specified.
Colorado	x		No. varies widely
Connecticut		x	

Table 4.6. Continued.

Department of Transportation	Yes	No	Specific Information
Florida	x		For large beams, cycle times of 3 days or less are recommended to eliminate shrinkage cracking. Therefore, Florida Department of Transportation (FDOT) limits release strengths to 80% of f'_c based on typical strength gain curves. Many prestressers still utilize different preapproved mixes and depending on time of year, project release requirements etc. may use the mix that produces the optimum turnaround time. It is not uncommon for a 5500 psi mix to break in the 7500 psi range.
Georgia	x		Probably so for 6000 psi concrete. For f'_c= 6000 psi concrete (design), actual strengths usually range from 7000 to 8000 psi
Illinois		x	
Iowa	x		Need to meet release strengths in 18 hours. Need to meet 28-day strength quickly so beams can be shipped early.
Kansas		x	Not done in Kansas due to the fact that Kansas has relatively poor aggregates; therefore, higher strengths are not easily achieved without a significant increase in cost.
Kentucky		x	
Louisiana		x	
Massachusetts		x	
Michigan		x	This is rarely a problem for HSC.
Minnesota	x		We have two methods in use. First is to use high early cement to obtain a high initial concrete strength, but final strengths then take much longer to achieve. The other method, as you described, does provide final strengths in excess of 10 ksi. No information on comparison of f'_{ci} and f'_c required.
Missouri	x		Of two bridges constructed, two sets of values for concrete strengths at transfer and at service fare as follows. Case 1: Specified f'_{ci}=5500 psi, Specified f'_c= 10000 psi, Actual f'_c=12300 psi. Case 2: Specified f'_{ci}=7500 psi, Specified f'_c=10000 psi, Actual f'_c=11400 psi. In contrast, projects currently in design or construction phase have specified f'_c of 500 to 1500 psi above f'_{ci}. Based on this we would not be surprised if we start seeing significantly higher f'_c than specified.
Montana	x		From approximately 300 tests for 28-day cylinder breaks from recent prestressed beams, the average strength was 9200 psi, median was 9300 psi, and the standard deviation was 1600 psi. It appears that the higher the transfer strength in a given amount of time, the lower percentage gain in final strength.
New Hampshire		x	We have not observed precasters designing mixes specifically to achieve a one day turnaround.
New Jersey	x		To assure that desired strengths are achieved New Jersey Department of Transportation specifies mix designs that ultimately produced higher strengths in service. Fabricators are awarded bonuses for good production and penalized for bad production. Mix proportion concrete strength approximately 10% higher than specified compressive strength.
New Mexico		x	No issues brought to us by prestress plant.
New York	x		Precasters generally use mix designs with expected 28 days strength 10 to 15% above what is required by the designs. Benefits: 1- Relatively early release of beds. 2- Allow shipping earlier than 28 days since girders could be shipped once compressive strengths are above the required minimum.

Table 4.6. Continued.

Department of Transportation	Yes	No	Specific Information
North Carolina	x		Precasters typically focus on achieving initial strengths (f'_{ci}) and acceptance strength (f'_c) by using high early cement and heat curing methods. Typically, f'_{ci} is achieved within 1 to 2 days and f'_c is achieved within 14 to 18 days. At acceptance strength, f'_c is usually 200 to 500 psi greater than the f'_c specified for designs. Unfortunately, no testing is done after acceptance. Therefore, 28 days strength is not known to compare to actual design strength.
North Dakota		x	The beams do not gain much strength after f'_{ci} has been reached.
Oklahoma	x		For f'_c less than and equal to 8000 psi (+/- 75%). For f'_c more than 8000 psi (+/- 70%).
Pennsylvania	x		We see that the transfer strength controls the design, so we use higher transfer and then higher 28 day.
Rhode Island	x		Typically higher transfer strengths are attained with accelerated curing systems heat/steam. High early strength mixes are known to attain lower strengths at 28 days than if cured under ambient conditions- the strength tends to flatten out at 7 days. It is difficult to list a correlation between the strength at release and the strength at service conditions.
South Carolina	x		Precasters overdesign their mix for faster production.
South Dakota	x		This occurs quite often. Fabricators who rely on radiant heat curing use mix designs with higher f'_c than fabricators who use steam curing. Unable to give more specific information.
Tennessee		x	We do not see this occurring too much on high strength girders, but it does tend to occur on normal strength girders.
Texas – Austin	x		Generally, 30 to 50% higher.
Vermont	x		Most of our prestressed structures are constructed with beams precasted under this scenario, especially with high strength transfer in short time frames. We have not made any analysis of what effect this has. The bridges seem to perform well.
Washington	x		Designs are controlled by f'_{ci} but high strength at release does not result in a significantly larger f'_c. Observation: f'_{ci} =7500 psi, then reduces slightly up to 7 days, then increases to about f'_c =10000 psi at 28 days.
Wisconsin		x	
Additional Respondents			
Texas - Houston	x		Information not available.
Structural Engineering Assoc.	x		Yes, in order to maintain their normal production schedule. There is no set pattern for overstrength.
Turner, Collie & Braden, Inc.	x		Yes, this occurs on a regular basis. We have observed variances of the order of 1500 psi increase in f'_c.

Twenty-two of the responding state DOTs observed that high initial strength requirements have led to an overstrength in f'_c at service. Positive responses indicate that in some cases (in general when $f'_{ci} > 6000$ psi) mixture designs are governed by the initial concrete stress. Thus, the specified release strength tends to be critical for prestressed concrete girder production.

In this study the definition of HSC is concrete with specified compressive strengths for design of 6000 psi or greater, made without using exotic materials or techniques (ACI 363, 1997). However, most of the responses indicate that high transfer strengths require special materials or techniques like accelerated curing. Two approaches are mentioned to obtain HSC. First is to obtain a high initial concrete strength (within 18 hours to 2 days) using high early cement and/or heat curing. In this case, the final strengths tend to level off quickly (around 7 days) and the strength gain is not significant. High early strengths obtained with accelerated curing methods (heat/steam) are known to attain lower strengths at 28 days than if cured under ambient conditions (see Rhode Island DOT response to question 9 in Table 4.6). Second is the common method of curing at ambient conditions, which tends to provide final strengths higher than those specified in designs. In this case, if precasters focus on achieving the high initial concrete strength demands (within 18 hours to 2 days) with ambient curing methods, then the specified 28-day strength is met quickly (before 28 days), so larger concrete strengths can be achieved at 28 days.

In particular, TxDOT's Austin office confirmed that the need for high transfer strengths has led to larger service strengths. They reported that the overstrength ranges from 30 to 50 percent relative to the specified f'_c. TxDOT's Houston District office also confirmed this trend, but information quantifying the overstrength was not available.

4.2.7 Concerns Related to the Use of HSC

Question 10 requests information about concerns related to the use of HSC prestressed bridge girders. In particular, this question is as follows.

Q 10: Please note any concerns you have related to the use of HSC prestressed bridge girders.

Table 4.7 identifies positive and negative responses for question 10 as well as some specific information given by the state DOTs.

Table 4.7. Concerns Related to the Use of HSC (Q 10).

Department of Transportation	Yes	No	Specific Information
Alabama		x	Have used HSC for several years without problems.
Alaska	x		All parameters in the design of HSC/HPC must be optimal to consistently provide satisfactory concrete strengths.
Arkansas		x	Prestressed bridge girders are not a predominant structure type in Arkansas.
California		x	
Colorado	x		With current technology it is difficult to take advantage of concrete strengths more than 9000 psi.
Connecticut		x	
Florida	x		FDOT typically utilizes HSC with 0.6 in. low lax diameter strands on a 2 in. grid which slightly violates the AASHTO minimum spacing between strands, but makes the best use of materials. In a few cases stress risers have occurred at the ends of long girders at release due to the large cambers. Various cushioning mechanisms have been utilized to solve this problem. In a few cases large camber growth has been a concern, requiring the beam to penetrate the deck slab at midspan.
Georgia	x		Still concerned about final camber.
Hawaii		x	
Illinois		x	The use of long spans is limited by transportation of girders. HSC may help to increase girder spacing and lower the number of girders.
Iowa	x		Predicting camber in HSC. Predicting losses. Transportation of long beams. Anchorage of reinforcement.
Kansas	x		None, other than the producer's ability to get the HSC.
Kentucky		x	
Louisiana	x		Initial cracking of girders during pouring and before release stage. We limited temperature to 160 ^{o}F max. During cold weather, steam is added, which tends to increase initial.
Massachusetts	x		Shrinkage and cracking, magnitude and size.
Michigan		x	No concerns with HSC in ranges less than 7000 psi.
Minnesota		x	
Mississippi		x	
Missouri	x		We are concerned in focusing on what is economically feasible and beneficial in Missouri. Striving for cost-saving designs and improved performance via HSC according to locally available materials.
Montana		x	
Nevada	x		Non-availability of suitable aggregates in the northern part of the state. There are not qualified precasters within the state.
New Hampshire	x		Specifications need to be upgraded for HSC.
New Jersey	x		Long-term quality control testing such as creep testing becomes a concern. We encourage fabricators to have mix designs pre-approved.
New Mexico		x	
New York	x		Since HSC has no criteria to control the penetration of chlorides when exposed to them, corrosion of steel is a problem. New York State Department of Transportation is moving to HPC with lower permeability. We are also using curing corrosion inhibitors and sealers.
North Dakota		x	
Ohio	x		Damage due to collision. Damage in grade separations.
Oklahoma		x	
Pennsylvania	x		For very high strengths, over 9000 psi, we are concerned about the applicability of the Specifications.

Table 4.7. Continued.

Department of Transportation	Yes	No	Specific Information
Rhode Island	x		Early tensile cracks at the transfer stress.
South Carolina	x		
South Dakota	x		Deflections, camber, and losses.
Texas – Austin		x	
Vermont	x		Brittle failure. Need for more prestress strands to take advantage of HSC. This then requires more steel to be added to already congested end beam detail.
Virginia		x	
Washington	x		Curing, overestimating losses, overestimating creep and camber.
Wisconsin		x	
Undisclosed DOT	x		
Additional Respondents			
Structural Engineering Associates	x		Service concrete strengths of up to 8500 psi are normally specified without problems. Fabricators would have difficulty with higher strengths. HSC at release can slow down production.
Turner, Collie & Braden, Inc.	x		Designs lead to longer turnarounds in the bed. Contractor loading girders too early results in erratic camber dimensions.

The responses indicate that almost one-half of the DOTs have some concerns related to the use of HSC. Some of the general concerns are discussed below.

4.2.7.1 Transportation of Larger Span Lengths

Maximum span lengths are limited by transportation of the girders. In these cases, HSC may be used to increase girder spacings. According to design recommendations (TxDOT Bridge Design Manual 2001a), maximum span lengths of prestressed concrete beams constructed economically can be up to 130 ft. for U54 Beams with girder spacing of 9.75 ft. using normal strength concrete and up to 130 ft. for Type IV beams (no value was found for the girder spacing) using NSC. However, design recommendations mentioned that a recent project in San Angelo utilized HSC with a concrete strength of 14000 psi to construct a 153 ft. span with Type IV beams (TxDOT 2001a). Moreover, the same document states that beams up to 150 ft. have been successfully transported, although at a premium cost.

4.2.7.2 Design Parameters for HSC

Design parameters in the AASHTO Specifications need to be upgraded for HSC of more than approximately 8500 psi. Several DOTs are reluctant to specify concrete compressive strengths at service (f'_c) higher than 8500 psi. The survey showed that the most popular range for the concrete strength at transfer (f'_{ci}) ranges from 4000 to 7000 psi, and at service typical f'_c values range from 5000 to 8500 psi. However, 15 percent of the DOTs utilize a higher concrete strength at service (10000 psi) for some cases, and 2 percent of the DOTs utilize a concrete strength at service of 12000 psi. The design equations in the AASHTO codes for prestressed concrete members are based on mechanical properties of NSC of 6000 psi or less. Information about the mechanical properties for HSC produced by Texas precasters can be found in Hueste et al. (2002 a,b).

4.2.7.3 Cracking

Initial cracking of girders during pouring and before the release stage is a concern. TxDOT practice indicates that cracking at release is not a problem since transfer is a temporary condition. Also, TxDOT engineers have provided input that this is not a major concern because if a crack occurs in the top of the beam at the end regions, it will close when the concrete slab is poured. More information on stress limits at transfer is available in Section 5.3.4.

4.2.7.4 Additional Concerns

Additional concerns include difficulties in providing 0.6 in. diameter strands at the proper spacing for some standard girder configurations. Also, research is needed to address critical issues, such as overestimation of losses and determination of creep, shrinkage, and camber for HSC. In some areas, suitable aggregates are not available, and in some cases, there are no qualified precasters to produce HSC prestressed girders.

4.2.8 Adjustments to Design Specifications for HSC Prestressed Bridge Girders

Question 11 of the survey requests information on adjustments applied to the specifications when designing HSC prestressed bridge girders.

> Q 11: Has your organization made any adjustment to the design specifications for HSC prestressed bridge girders based on research findings (such as in the allowable stresses of resistance factors?
> If so, please describe and provide a reference to relevant research, if available.

Table 4.8 identifies positive and negative responses for this question, as well as some specific information given by the respondents.

The survey indicates that most of the DOTs have not made adjustments to the design specifications for HSC prestressed bridge girders. Of the seven DOTs that have modifications, Minnesota and South Dakota have modified the equation for the modulus of elasticity, and Washington has modified the allowable stresses and equations for losses, creep, and camber based on in-house practice. Louisiana is conducting research that will be completed in 2003, and it is expected that the allowable stresses will change based on these findings.

Table 4.8. Adjustments to Design Specifications for HSC Prestressed Bridge Girders (Q 11).

Department of Transportation	Yes	No	Specific Information
Alabama		x	Developed some HPC mix designs for a HPC showcase project.
Alaska		x	
Arkansas		x	
California		x	
Colorado		x	
Connecticut		x	
Florida		x	
Georgia		x	
Hawaii		x	
Idaho		x	
Illinois		x	
Iowa		x	
Kansas	x		Reduced the allowable tension in the precompressed tensile zone caused by the prestressing force, service loads and prestressed losses to $0.125\sqrt{f'_c}$ ($3.95\sqrt{f'_c}$ in psi units). This Kansas Department of Transportation (KsDOT) policy is for fatigue considerations should cracking of the beam occur.

Table 4.8. Continued.

Department of Transportation	Yes	No	Specific Information
Kentucky		x	
Louisiana	x		We have developed special provisions for our HPC projects based on our sponsored research. Our current research will be completed in 2003. We hope to change allowable stresses based on the 2003 research.
Massachusetts		x	
Michigan		x	
Minnesota	x		The only modification in design is the method to calculate the modulus of elasticity "E_c". We use the equation developed by the University of Minnesota for our high-strength mixes.
Mississippi		x	
Missouri		x	No, but a research study currently underway with the University of Missouri-Rolla, R100-002, is intended to provide results which will validate or recommend design assumptions for HPC.
Montana		x	
Nevada		x	
New Hampshire		x	
New Jersey		x	
New Mexico		x	New Mexico State University did some prestress loss measurements using fiber optics. Losses were within design assumptions.
New York		x	
North Carolina		x	
North Dakota		x	
Ohio		x	
Oklahoma		x	
Pennsylvania		x	
Rhode Island		x	
South Carolina		x	
South Dakota	x		Modification of the method to compute the modulus of elasticity.
Tennessee		x	
Texas – Austin		x	
Vermont	x		Our specifications were developed regionally with neighboring states. Contact the New England region of PCI for more info.
Virginia	x		Not using LRFD
Washington	x		Not based on research findings but based on in-house practice. Modification of creep equation, modification of methods to compute losses, camber, and modification of the allowable stresses. Design memorandums (concrete density, shear, bursting, etc.)
Wisconsin		x	
Additional Respondents			
Texas – Houston		x	
Structural Engineering Associates		x	
Turner, Collie & Braden, Inc.		x	We are using TxDOT or C DOT standard practice.

4.3 PART II: DESCRIPTION OF TYPICAL BRIDGES WITH HSC PRESTRESSED BRIDGE MEMBERS

4.3.1 General

Part II of the survey of current practice focused on determining basic characteristics of typical bridges with HSC prestressed bridge girders used by the state DOTs. More specifically, the content of Part II of the questionnaire is as follows.

<u>Part II: Description of Typical Bridges with HSC Prestressed Bridge Members</u>

In the following table (Table 4.9), please provide the following information based on the practices of your organization.

- Indicate the types of bridges for which HSC prestressed bridge girders have been used by your organization.
- Provide the ranges for span length and concrete compressive strength (f'_c), for each structural type selected.
- Note how prevalent each type is for HSC prestressed bridge members, by filling in the percentage column.

Table 4.9. Typical Bridges with HSC Prestressed Bridge Members.

Span Type	**Structural Type**	**Span** (range in ft.)	f'_c (range in psi)	**Percentage**
Simple Span	Slab			
	Voided Slab			
	Double T			
	Closed Box CIP			
	AASHTO Beam			
	Bulb			
	Box Girder			
	Other (describe)			
Continuous Span*	Slab			
	Voided Slab			
	AASHTO Beam			
	Post-tensioned AASHTO			
	Beam			
	Bulb			
	Box			
	Other (describe)			

* For this study, the term "continuous span" refers to the case where the girders are continuous over a support. When continuity is provided within the cast-in-place slab only, this is considered a "simple span."

The information requested regarding prevalence of each structural type for HSC prestressed bridge members seems to have been interpreted in different ways. However, the reported values are included in the following tables for completeness. In addition, although an f'_c less than 6000 psi was not considered to be HSC for this study, some lower strength values were reported and are included in the results reported below.

Figure 4.1 shows the prevalence of different types of prestressed concrete bridges. It is evident that the AASHTO and bulb beams are the most predominant among all the states.

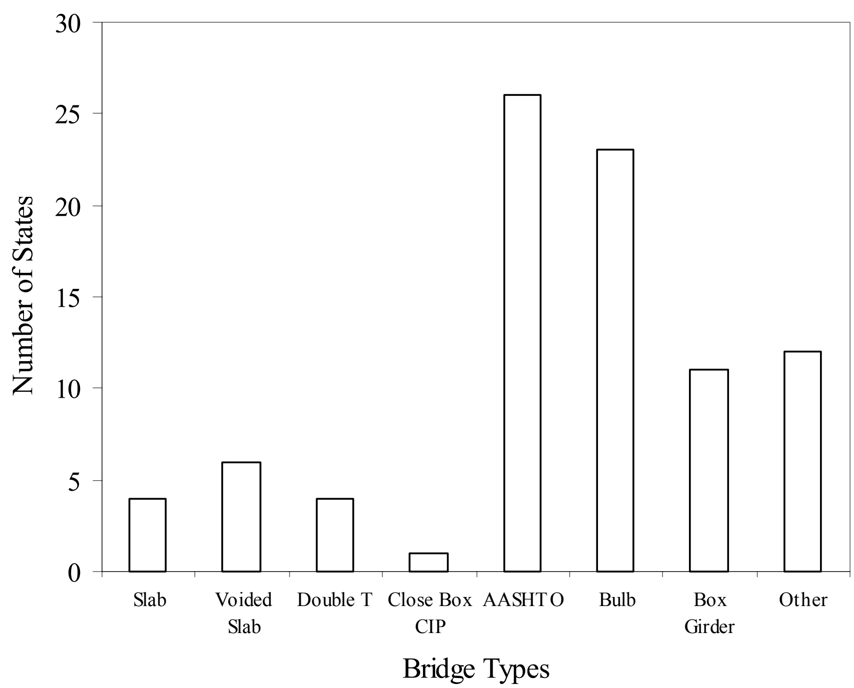


Figure 4.1. Prevalence of Bridge Types with HSC Prestressed Girders.

The most popular HSC girder type among the responding DOTs is the AASHTO beam (26 states), followed by the bulb (23 states) and the box girder (11 states). Voided slabs (6 states), slabs (4 states), double T-beams (4 states), and closed box cast-in-place (CIP) beams (1 state) are the structural types with less use, although the CIP closed box is used for long spans (typically up to 150 ft.). It should be noted that the Texas U-beams are used not only in Texas, but also in Colorado and New Mexico.

Tables 4.10 and 4.11 summarize the findings for typical ranges for span lengths and concrete compressive strength, for each type of bridge surveyed as described above.

Table 4.10. Typical Range for Span Lengths by Structural Type.

Structural Type	**Span Length Range** (ft.)			
	30-60	60-90	90-120	120-150
Slab	x			
Voided Slab	x			
Double T	x			
Closed Box CIP*	x	x	x	x
AASHTO	*	x	x	x
Bulb	*	x	x	x
Box Girder	*	x	x	*
Other (U-beam)	*	x	x	x

* Rarely used

Table 4.11. Typical Range for Concrete Compressive Strengths by Structural Type.

Structural Type	**Concrete Compressive Strength Range** (ksi)				
	3.5-6	6-8	8-10	10-12	14
Slab	x	*	*		
Voided Slab	x	*	*		
Double T	x	*			
Closed Box CIP*	x				
AASHTO	*	x	x	*	* (one case)
Bulb	*	x	x		
Box Girder	*	x			
Other (U-beam)		x	x	*	

* Rarely used

4.3.2 Shorter Spans

Slab, voided slab, and double T-beams are more prevalent for shorter span lengths. Tables 4.12 through 4.14 show the ranges for span length and f'_c for these structural types, as reported by the DOTs. In addition, the reported prevalence of each type for HSC prestressed bridge members is provided, although DOTs appear to have interpreted the question differently. The typical range for shorter span lengths is from approximately 30 to 60 ft. and the typical range for f'_c varies from approximately 3500 to 6000 psi. An f'_c of 8000 psi was also reported. However, Table 4.13 shows a case where the New York DOT uses voided slabs for beams spanning up to 100 ft. with f'_c up to 10000 psi.

Table 4.12. Typical Bridges with HSC Prestressed Members - Structural Type: Slab.

Department of Transportation	Span (ft.)				*f'c* (ksi)								Prevalence of HSC (%)
	20-30	30-40	40-50	50-60	3	4	5	6	7	8	9	10	
California	x	x			x	x	x						5
Colorado	x	x				x							0
Florida	x	x					x	x					5
Hawaii	x	x						x	x				20
Illinois		x				x	x						100
Montana	x					x	x						0
New York	x								x	x	x	x	1.5
Texas – Austin		x						x	x				0.4
Vermont	x				x	x							5
Virginia	x	x	x			x							-
Washington	x	x	x	x		x	x						0
Total	9	8	2	1	2	7	5	3	3	1	1	1	
Additional Respondents													
Structural Eng. Associates		x	x				x	x					10
Turner, Collie & Braden		x	x				x						0

Table 4.13. Typical Bridges with HSC Prestressed Members - Structural Type: Voided Slab.

Department of Transportation	Span (ft.)							f'_c (ksi)								Prevalence of HSC (%)
	20-30	30-40	40-50	50-60	60-70	70-80	80-110	3	4	5	6	7	8	9	10	
Alaska		x										x				10
California	x	x	x					x	x	x	x					5
Idaho	x	x	x							x	x					90
Illinois			x	x	x	x			x	x						100
New York		x	x	x	x	x	x					x	x	x	x	20
North Carolina		x	x							x	x	x	x			20
Vermont		x	x	x				x	x							62
Virginia	x	x	x						x							
Washington	x	x	x	x	x				x	x	x					60
Total	4	8	8	4	3	2	1	2	5	5	4	3	2	1	1	
Additional Respondents																
Turner, Collie & Braden		x								x	x					20

Table 4.14. Typical Bridges with HSC Prestressed Members - Structural Type: Double T.

Department of Transportation	Span				f'_c (ksi)						Prevalence of HSC (%)
	20-30	30-40	40-50	50-60	3	4	5	6	7	8	
California		x	x	x	x	x	x	x			5
Minnesota	x	x	x	x				x	x		-
Oklahoma		x	x	x						x	5
Texas – Austin		x	x	x				x	x		0.5
Vermont		x	x		x	x					12
Total	1	5	5	4	2	2	1	3	2	1	
Additional Respondents											
Turner, Collie & Braden			x				x	x			

4.3.3 Larger Spans

Closed box CIP beams, AASHTO beams, bulb beams, and box beams are more prevalent for longer span lengths. Tables 4.15 through 4.18 show the ranges for span length and concrete compressive strength for these structural types, as reported by the DOTs. In addition, the reported prevalence of each type for HSC prestressed bridge members is provided, although this value seems to have different interpretations among the respondents. The typical range for larger span lengths is from approximately 60 to 150 ft., and the typical range for specified concrete strengths at service (f'_c) varies from approximately 6000 to 10000 psi. An f'_c of 13000 psi was also reported.

Table 4.15 shows that the CIP closed box beams can be used for spans from 50 to 150 ft. with f'_c of 6000 psi, but they are rarely used. Table 4.16 shows that the AASHTO Beams are used for a variety of span lengths. However, the typical range for span lengths is approximately 75 to 130 ft. and the typical range for f'_c is approximately 6000 to 8000 psi. In particular, the prevalent range for span length is 100 to 120 ft. It was also reported that span lengths up to 155 ft. and f'_c of 14000 psi can be used. Specifically, this case was reported by TxDOT's Austin office.

Table 4.15. Typical Bridges with HSC Prestressed Members - Structural Type: Closed Box CIP.

Department of Transportation	Span (ft.)						f'_c (ksi)			Prevalence of HSC (%)
	50-60	60-80	80-100	100-120	120-130	130-140	4	5	6	
California[1]		x					x	x		70
Colorado[2]	x						x	x	x	0
Washington	x	x	x	x	x	x	x	x		0
Total	2	2	1	1	1	1	3	3	1	

[1] CA DOT reported span lengths up to 600 ft.

[2] CO DOT reported span lengths up to 200 ft.

Table 4.16. Typical Bridges with HSC Prestressed Members - Structural Type: AASHTO Beam.

Department of Transportation	Span (ft.)							f'_c (ksi)							Preva-lence of HSC (%)
	40-60	60-80	80-100	100-120	120-140	140-150	150-160	5	6	7	8	9	10	11-14	
Alabama			x	x					x	x	x				40
California[1]	x	x	x	x				x	x	x					10
Florida			x	x				x	x	x	x				35
Georgia		x	x	x					x	x	x	x	x		30
Hawaii	x	x	x	x	x				x	x					80
Idaho	x	x	x	x				x	x						90
Illinois[2]	x	x	x					x	x						small
Kansas				x					x						30
Kentucky					x			x	x	x					5
Louisiana		x	x	x	x								x		2
Michigan				x					x	x					50
Minnesota	x	x	x	x	x	x	x		x	x	x				95
Mississippi		x	x	x					x						50
Montana	x	x	x	x	x	x		x	x	x					70
New Hampshire		x	x					x	x	x	x				10
New Jersey[3]			x	x	x	x	x		x	x	x				
New Mexico			x	x					x	x	x	x	x		30
New York		x	x	x	x					x	x	x	x		1
North Carolina	x	x	x	x	x			x	x	x	x				70
North Dakota				x	x	x			x	x					10
Ohio		x	x	x	x	x	x			x					45
Oklahoma	x	x	x	x							x	x	x		85
Pennsylvania		x	x	x	x	x			x	x	x				50
South Dakota	x	x	x						x	x	x				50
Texas - Austin			x	x	x	x			x	x	x	x	x	x	76.7
Vermont[4]	x	x	x					x							5
Virginia	x	x	x						x	x	x				-
Wisconsin	x	x	x	x				x							30
Total	12	19	24	22	12	7	3	10	22	20	14	5	6	1	

Table 4.16. Continued.

Department of Transportation	Span (ft.)							f'_c (ksi)							Preva-lence of HSC (%)
	40-60	60-80	80-100	100-120	120-140	140-150	150-160	5	6	7	8	9	10	11-14	
Additional Respondents															
Texas - Houston		x	x	x				x	x	x	x				50
Structural Engineering Associates	x	x	x	x	x			x	x	x	x				85
Turner, Collie & Braden, Inc.	x	x	x	x	x			x	x	x	x	x			80

[1] CA DOT reported f'_c = 4 ksi
[2] IL Beam
[3] WA DOT reported span lengths up to 222 ft.
[4] VT DOT reported f'_c = 4 ksi

Table 4.17 shows that the bulb beams are also used for a wide range of span lengths. However, the most typical range for span lengths is from approximately 95 to 135 ft., and the typical range for the specified concrete strength (f'_c) is from approximately 6000 psi to 8000 psi. In particular, the most typical span length is 115 ft., followed by the span of 135 ft. Specifically in Texas, TxDOT's Austin and Houston offices do not use bulb beams, although this girder type is used by some organizations located in Texas.

Table 4.18 shows that box girders are also used for large range of span lengths, although they are not widely used. The most typical range for span lengths is from approximately 55 to 115 ft., and the typical range for the concrete strength (f'_c) is from approximately 6000 to 8000 psi. TxDOT's Austin and Houston offices collectively use the box girder section for span lengths ranging from 55 to 115 ft. with f'_c values from 6000 to 8000 psi.

Table 4.17. Typical Bridges with HSC Prestressed Members - Structural Type: Bulb.

Department of Transportation	Span (ft.)							f'_c (ksi)							Prevalence of HSC (%)
	40-60	60-80	80-100	100-120	120-140	140-150	150-160	4	5	6	7	8	9	10	
Alabama				x	x					x	x	x			60
Alaska		x	x	x	x					x	x				90
California			x	x	x	x		x	x	x	x				10
Colorado	x	x	x	x	x	x	x			x	x	x	x		34
Florida				x	x	x				x	x	x			10

Table 4.17. Continued.

Department of Transportation	Span (ft.)							f'_c (ksi)							Prevalence of HSC (%)
	40-60	60-80	80-100	100-120	120-140	140-150	150-160	4	5	6	7	8	9	10	
Georgia			x	x	x					x	x	x			70
Idaho	x	x	x	x	x					x	x				90%
Illinois			x	x	x	x			x	x					small
Iowa	x	x	x	x	x				x	x	x	x	x		15%
Kansas				x	x	x				x	x	x			100%
Massachusetts			x	x								x			50%
Michigan					x	x				x	x				20%
Mississippi						x				x					100%
Missouri				x							x				2 bridges
Montana	x	x	x	x	x				x	x	x				70%
New Hampshire		x	x	x					x	x	x	x			88%
New Mexico			x	x	x					x	x	x	x	x	30%
New York		x	x	x	x						x	x	x	x	1%
North Carolina			x	x	x				x	x	x	x			80%
Ohio					x	x	x				x				5%
Oklahoma				x	x							x	x	x	10%
Virginia	x	x	x							x	x	x			
Washington	x	x	x	x	x	x	x					x	x	x	100%
Wisconsin				x	x	x			x						70%
Total	6	9	15	20	19	10	3	1	7	17	18	13	6	4	
Additional Respondents															
Structural Engineering Associates					x						x	x	x		100%
Turner, Collie & Braden, Inc.	x	x	x	x	x						x	x			90%

Tables 4.19 and 4.20 show other types of beams that are used for a variety of span lengths and concrete strengths. These beams are specific for one or more states and are not widely used. Among these types of beams are the tri-deck, inverted T, side-by-side box beams, Missouri beams, Minnesota beams, and Texas U-beams.

Table 4.18. Typical Bridges with HSC Prestressed Members - Structural Type: Box Girder.

Department of Transportation	Span (ft.)							f'_c (ksi)					Prevalence of HSC (%)
	40-60	60-80	80-100	100-120	120-140	140-150	150-160	4	5	6	7	8	
California				x	x	x		x	x	x	x		5
Colorado	x	x	x	x	x				x	x	x	x	10
Florida					x	x						x	5
Idaho		x							x	x			90
Kentucky			x						x	x	x		10
Massachusetts			x	x								x	50
Ohio	x	x							x	x	x		50
Pennsylvania				x						x	x	x	50
Rhode Island	x	x	x							x	x		85
Texas - Austin			x	x						x	x	x	5.70
Vermont	x	x						x	x	x			16
Washington						x	x	x	x				0
Total	4	5	5	5	3	3	1	3	7	9	7	5	
Additional Respondents													
Texas - Houston	x	x	x	x					x	x	x	x	5
Structural Engineering Associates	x	x	x	x					x	x	x	x	70

Table 4.19. Typical Bridges with HSC Prestressed Members - Structural Type: Other - Span Lengths.

Department of Transportation	Girder Type	Span (ft.)								Prevalence of HSC (%)
		40-60	60-80	80-100	100-110	110-120	120-130	130-150	150-200	
Colorado*	U-beam	x	x	x	x	x	x	x	x (200)	2
Idaho	Tri-deck	x								90
Kansas	Inverted T	x	x	x						100
Michigan	Side-by-Side Box Beams				x	x	x	x		30
Minnesota	Prestressed Rect. Beam	x								5
Missouri	MO beam	x	x							3 bridges
Montana	Tri-deck	x								70
New Mexico	U-beam			x	x	x	x			40
New York*	Channel Bridge					x	x	x	x (165)	0.50
South Dakota	MN beam			x	x	x	x			50
Texas - Austin	U-beam				x	x	x			
Washington	Deck bulb T	x	x	x	x	x	x	x	x (160)	16.7
Total		7	4	5	6	7	7	4	3	

Table 4.19. Continued.

Department of Transportation	Girder Type	Span (ft.)								Prevalence of HSC (%)
		40-60	60-80	80-100	100-110	110-120	120-130	130-150	150-200	
Additional Respondents										
Texas - Houston	U-beam			x	x	x	x			75
Turner, Collie & Braden, Inc.	U-beam					x	x			80

*Colorado DOT and New York DOT reported span lengths up to 200 and 165 ft., respectively.

Table 4.20. Typical Bridges with HSC Prestressed Members - Structural Type: Other – Concrete Strengths.

Department of Transportation	Girder Type	f'_c (ksi)							Prevalence of HSC (%)
		5	6	7	8	9	10	11-12	
Colorado	U-beam	x	x	x	x	x			2
Idaho	Tri-deck	x	x						90
Kansas	Inverted T			x	x				100
Michigan	Side-by-side-Box Beams	x	x	x					30
Minnesota	PS Rect. Beam		x	x	x				5
Missouri	MO beam			x	x	x	x		3 bridges
Montana	Tri-deck	x	x	x					70
New Mexico	U-beam				x	x	x	x	40
New York	Channel bridge			x	x	x	x		0.50
South Dakota	MN beam		x	x	x				50
Texas - Austin	U-beam		x	x	x				
Washington	Deck bulb T	x	x	x	x				16.7
Total		5	8	10	9	4	3	1	
Additional Respondents									
Texas - Houston	U-beam	x	x	x	x				75

In particular, the Texas U-beam is being used not only in the state of Texas but also in other states such as Colorado and New Mexico. In this case, the typical range for the span lengths is from approximately 75 to 140 ft., and f'_c ranges from approximately 6000 to 10000 psi, although New Mexico uses an f'_c up to 12000 psi.

5 OUTLINE OF PARAMETRIC STUDY AND ANALYSIS PROCEDURES

5.1 GENERAL

Researchers conducted a parametric study for single-span prestressed concrete bridge girders to mainly investigate the controlling limit states for various concrete strengths using both the AASHTO Standard and LRFD Specifications (2002 a,b). The effects of changes in concrete strength, strand diameter, girder spacing, and span length were also considered. This study focused only on limit states related to flexure for service and ultimate conditions, and additional design limit states were not evaluated. TxDOT currently uses an HS25 truck loading for a number of designs. The loading used in this study was based on the specified loads in the AASHTO specifications, which reference an HS20 truck loading. To carry out the parametric study, four subtasks were performed, as follows:

1. Develop spreadsheets to perform iterative design and analysis calculations.
2. Evaluate several case study bridges with U54 and Type IV beams designed using the AASHTO Standard Specifications and compare the results with those from TxDOT prestressed concrete bridge girder design program, PSTRS14 (TxDOT 1980). This comparison is needed to check the procedures and equations used in this study with those of TxDOT, based on the AASHTO Standard Specifications, as well as to ensure consistency between results. Two case study designs using U54 Beams and two case study designs using Type IV beams are presented.
3. Define analysis and design assumptions and design variables for the parametric study.
4. Perform the analysis for the parametric study.

This section describes the girder sections considered, the analysis and design assumptions, the design parameters, and comparison of the analysis procedure to current TxDOT practices. The results of the parametric study are reported in Sections 6 and 7.

5.2 GIRDER SECTIONS

5.2.1 U54 BEAMS

TxDOT began development of the Texas precast concrete U54 beam in the mid 1980s to create an alternative to the AASHTO Type IV and Texas Type C precast concrete I sections. It was not created to replace the precast concrete I-beam but to satisfy aesthetic demands with economy and ease of construction (TxDOT 2001a). Since 1993, when the first U-beam was constructed in Houston, research has been conducted to study the behavior of the U-beams. Two U-beam sections for use as prestressed concrete bridge girders, U40 and U54, were developed. The TxDOT U54 beam is trapezoidal in cross-section and is open at the top with two flanged stems. The depth of the U54 beam is 54 in. with a total width at the top of the stems of 96 in., a thickness of 5 in. per web, and the bottom flange thickness can accommodate three rows of strands. Figure 5.1 shows the configuration and dimensions of the U54 beam cross-section. The U40 section is similar in shape, with a depth of 40 in. For normal beam concrete strengths and 0.5 in. diameter strands, the recommended economical span length limit is 110 ft. (girder spacing = 7.5 ft.) for the U40 beam and 130 ft. (girder spacing = 9.75 ft.) for the U54 beam (TxDOT 2001a). The U54 section was selected for this study because of the focus on long span bridges.

5.2.2 TYPE IV BEAMS

The Type IV beam is an AASHTO Standard beam and has been the dominant beam since 1986. This beam is considered as a tough stable beam, and it is recommended for span lengths up to 130 ft. for normal concrete strengths and 0.5 in. diameter strands (TxDOT 2001a). However, Type IV beams with a span of 153 ft. were constructed as a part of a project in San Angelo, Texas, where HSC of 14000 psi was used (TxDOT 2001a). The Type IV beam is an I cross-section with two flanges. The depth of the Type IV beam is 54 in. with a 20 in. wide top flange and 26 in. wide bottom flange. This beam section has an 8 in. thick web, and the thickness of the top and bottom flanges are 8 in. plus a variable section of 6 in. high. Figure 5.2 shows the configuration and dimensions of the Type IV beam cross-section.

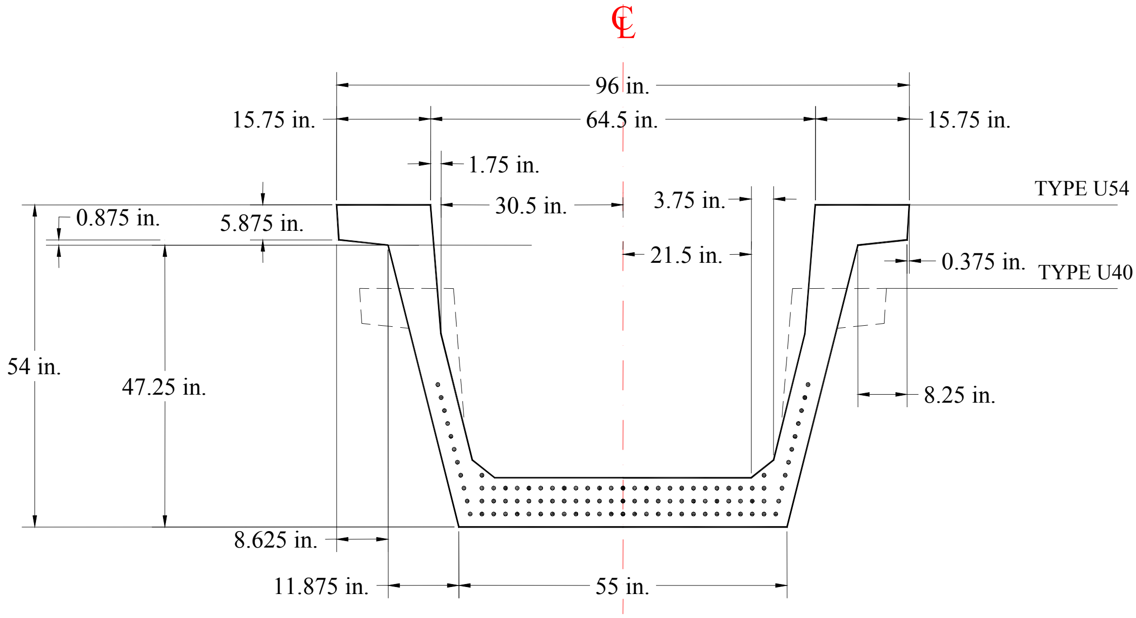


Figure 5.1. Configuration and Dimensions of the TxDOT U-Beam (adapted from TxDOT 2001b).

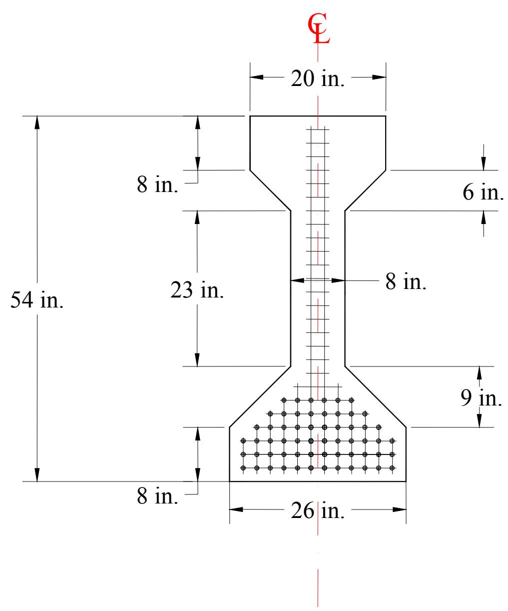


Figure 5.2. Configuration and Dimensions of the Type IV Beam (adapted from PCI 1997).

5.3 ANALYSIS AND DESIGN ASSUMPTIONS

This section addresses the analysis and design assumptions for prestressed concrete bridge girders based on the AASHTO Standard and LRFD Specifications. The Preliminary Design Guide for U-Beam Bridges (TxDOT 2001b) as well as the TxDOT Bridge Design Manual (TxDOT 2001a) were referenced.

5.3.1 Analysis

In both the AASHTO Standard and LRFD Specifications, the design of prestressed concrete bridge girders is based on the calculation of stresses under service loads and their comparison with specified allowable stresses. Allowable stresses in concrete are specified to control flexural compressive and tensile stresses in extreme fibers at any section along the member. Thus, the philosophy of prestressed concrete design resides in the emphasis on serviceability conditions. Member proportioning and prestressing strand layouts are selected to satisfy the serviceability criteria, and the ultimate capacity is seen as an additional, although essential, condition that is verified at a later stage.

5.3.2 Design Assumptions

Some designs (including TxDOT designs) are selected to minimize the number of strands required for a given span length, cross section, and girder spacing; then the corresponding required concrete strengths at release and final conditions of service are found. For the parametric study, the required minimum number of strands for a given span length and beam spacing were also determined, but the concrete strength (f'_c) was set at certain value with an initial concrete strength (f'_{ci}) of 0.75 f'_c. In this way, designs for different f'_c values can be compared to better understand the benefit and sensitivity of varying f'_c. The initial concrete strength was allowed to vary up to f'_c as PSTRS14 (TxDOT 1980) does in cases where necessary. Thus, it is assumed that HSC can be achieved at the time corresponding to release, which is typically at 1 day of age at a precast plant.

Prestress losses were computed based upon the detailed method, where elastic shortening, relaxation, shrinkage, and creep were estimated separately. Estimation of the prestress losses may be carried out at several levels. In common practical design cases, a lump sum estimate is sufficient. For this study, where accuracy was required, the detailed analysis was used to estimate separate losses that take into account the member geometry, material properties, environmental conditions, and construction method. The equations used for the computation of losses at transfer and at service conditions for designs using both the Standard and LRFD Specifications are shown in Section 3.3. In designs under the Standard Specifications, however, an approximation was made in the computation of loss due to relaxation at transfer in order to match the TxDOT procedure. In the Standard Specifications, an equation is provided for the total loss due to relaxation. TxDOT considers approximately one-half of this total amount for both losses at transfer and after transfer. Therefore, in the parametric study the same approach was used. In the LRFD Specifications, prestress loss due to relaxation at transfer is function of the time estimated in days from stressing to transfer. Researchers used 1 day for the time variable, as this is usually the time at which the girders are released from the forms.

5.3.3 Service Stresses for Flexural Design

This section provides the equations used to compute the stresses at the extreme top and bottom fibers of the precast sections in unshored composite construction for a simply supported beam. According to the conventional flexural design of composite beams (Nawy 2000) and the allowable stresses given in the AASHTO Standard and LRFD Specifications, three different stages were considered to compute the flexural stresses plus an additional check of the compressive stress. These stresses are used in the design to verify that the allowable stresses are not exceeded for service conditions. The sign convention for the resulting stresses is positive for tension and negative for compression.

5.3.3.1 Initial Stage

For the initial stage, the initial prestressing force (immediately after transfer) is applied to the precast section (non-composite section) before the concrete slab is cast. Initial losses in prestress are those that occur during and immediately after transfer of prestress.

$$f_t = \frac{-P_i}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{M_D}{S_t} \tag{5.1}$$

$$f_b = \frac{-P_i}{A}\left[1 + \frac{ec_b}{r^2}\right] + \frac{M_D}{S_b} \tag{5.2}$$

where:

f_t , f_b	=	Concrete stresses at top and bottom extreme fibers, respectively
P_i	=	Initial prestressing force
M_D	=	Moment due to self-weight
e	=	Eccentricity of tendons from the concrete section center of gravity (cgc)
A	=	Cross-sectional area of the precast beam
c_b , c_t	=	Distances from the cgc to the extreme top and bottom fibers, respectively, for the precast section alone
r	=	Radius of gyration
S_t , S_b	=	Section moduli of the precast section alone, referencing the extreme top and bottom fibers of the precast section, respectively

5.3.3.2 Intermediate Stage

For the intermediate stage, the effective prestressing force (after losses) plus the total dead load are acting on a composite section, after the cast-in-place concrete slab hardens. The prestress losses at this stage include the initial losses plus all time-dependent losses (same losses at the final stage).

$$f_t = \frac{-P_e}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{(M_D + M_{SD})}{S_t} - \frac{M_{CSD}}{S_{Ct}} \tag{5.3}$$

$$f_b = \frac{-P_e}{A}\left[1 + \frac{ec_b}{r^2}\right] + \frac{(M_D + M_{SD})}{S_b} + \frac{M_{CSD}}{S_{Cb}} \tag{5.4}$$

where:

P_e	=	Effective prestressing force
M_{SD}	=	Moment due to superimposed dead load (cast-in-place slab and diaphragms) applied prior to composite action between the girders and slab
M_{CSD}	=	Moment due to superimposed dead load (rail weight) applied after composite action between the girders and slab
S_{Ct}, S_{Cb}	=	Section moduli of the composite section, referencing the extreme top and bottom fibers of the precast section, respectively

5.3.3.3 Final Stage

For the final stage, the effective prestressing force (after total losses) plus the total dead load and total live and impact loads are acting on a composite section, after the cast-in-place concrete slab hardens. Total losses include the initial losses plus all time-dependent losses. However as noted earlier, for the Service III limit state in the LRFD Specifications, only 80 percent of the live and impact load must be considered in Equation 5.6.

$$f_t = \frac{-P_e}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{(M_D + M_{SD})}{S_t} - \frac{(M_{CSD} + M_{L+I})}{S_{Ct}} \qquad (5.5)$$

$$f_b = \frac{-P_e}{A}\left[1 + \frac{ec_b}{r^2}\right] + \frac{(M_D + M_{SD})}{S_b} + \frac{(M_{CSD} + M_{L+I})}{S_{Cb}} \qquad (5.6)$$

Where M_{L+I} is the moment due to live and impact load.

5.3.3.4 Additional Check of Compressive Stresses

The additional check of the compressive stress evaluates the compressive stress due to the total live loads plus one-half of the sum of the compressive stresses due to effective prestress and the total dead loads. The compressive stress at the top fiber is found, as follows.

$$f_t = \frac{1}{2}\left(\frac{-P_t}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{(M_D + M_{SD})}{S_t} - \frac{M_{CSD}}{S_{ct}}\right) - \frac{M_{L+I}}{S_{ct}} \qquad (5.7)$$

5.3.4 Stresses at Transfer

Research shows that longer spans can be achieved using HSC. However, because the additional capacity in some cases comes at the expense of higher initial concrete strength requirements at transfer, stresses at release are critical. Allowable temporary tensile and compressive stresses for the AASHTO Standard and LRFD Specifications are shown in Table 5.1. In the parametric study, the highest temporary allowable tensile stresses were used, as shown in Table 5.2.

Table 5.1. Allowable Stresses Specified by the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b).

Type of Stress		Allowable Stresses
Initial Stage: Immediately After Transfer (After Initial Loss in the Prestressing Force)	Tension	200 psi or $3\sqrt{f'_{ci}}$ *
	Compression	$0.6f'_{ci}$
Intermediate State: After Cast-in-Place Concrete Slab Hardens. Only Sustained Loads. (After Final Loss of Prestressing Forces)	Compression	Standard: $0.4f'_c$ LRFD: $0.45f'_c$
Final Stage: Total Dead and Live Loads (After Final Loss of Prestressing Forces)	Tension	$6\sqrt{f'_c}$
	Compression	$0.6f'_c$
Additional Check of the Compressive Stress at the Final Stage	Compression	$0.4f'_c$

Notes:

f'_c and f'_{ci} are in psi

* When the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete. The maximum tensile stress shall not exceed $7.5\sqrt{f'_{ci}}$ for the Standard Specifications and 6.96 $\sqrt{f'_{ci}}$ ($0.22\sqrt{f'_{ci}}$ in ksi) for the LRFD Specifications.

Table 5.2. Allowable Stresses during Transfer Used in the Parametric Study.

	Standard Specifications	LRFD Specifications
Compression (psi)	$0.6f'_{ci}$	$0.6f'_{ci}$
Tension (psi)	$7.5\sqrt{f'_{ci}}$	$6.96\sqrt{f'_{ci}}$

Note: f'_c and f'_{ci} are in psi

The maximum tensile stress at transfer of $7.5\sqrt{f'_{ci}}$ was used for designs using the Standard Specifications to be consistent with the TxDOT prestressed concrete bridge design program, PSTRS14. AASHTO allows this larger stress value when additional bonded reinforcement is provided to resist the total tension force in the concrete when the tensile stress exceeds $3\sqrt{f'_{ci}}$, or 200 psi. The maximum tensile stress of $6.96\sqrt{f'_{ci}}$ was also taken for designs based on the AASHTO LRFD Specifications. Similar to the Standard Specifications, the LRFD Specifications allow a larger tensile stress when additional bonded reinforcement is provided to resist the total tension force in the concrete when the tensile stress exceeds $3\sqrt{f'_{ci}}$, or 200 psi.

Based on input from TxDOT engineers, there are several reasons why the use of the larger tensile stress limit at transfer ($7.5\sqrt{f'_{ci}}$) used by the PSTRS14 program (TxDOT 1980) is reasonable:

1. PSTRS14 conservatively assumes that the strands develop instantaneously at the end of the girder when actually it takes about 60 bar diameters in length from the end of the beam for the strand to fully transfer the prestressing force.
2. Some bonded reinforcement is provided at the beam end near the top face, so it is justifiable to adopt the higher limiting value for the initial stresses.
3. There is a large end block area that is typically not taken into account in the computation of flexural stresses at the end of the beam. In general, designs only consider the section of the beam in the end regions, and the only available concrete to resist tension is that in the top flange (two small flanges in the U54 beam), when actually, the entire solid section of the beam is available to resist the forces at that section. Thus, the tensile stress at the top fiber is lower than computed at the beam ends.
4. Finally, the high tensile stress produced at the beam ends during transfer is a temporary condition. If a crack occurs in the top of the beam at the end regions, it will close when the concrete slab is poured. Thus, in the final condition, the tensile stresses at the end of the beam will be reduced.

For these reasons, and to be consistent with TxDOT practices, the allowable stresses at the beam ends at transfer for the Standard and the LRFD Specifications were taken as those shown in Table 5.2 for this study.

5.3.5 Ultimate-Strength for Flexural Design

The flexural moment capacity provided by the prestressed concrete composite beam was initially computed using the equations provided by the AASHTO Standard and LRFD specifications. However, researchers found an inconsistency when computing the depth of the equivalent rectangular stress block (a) for the case of a flanged section. The inconsistency occurs when assuming a rectangular section first, and 'a' is found to be greater than the thickness of the slab (h_f). The inconsistency occurs when recomputing the 'a' value using equations for a flanged section, and a value smaller than h_f or even a negative value is computed. This is due to the fact that in calculating the 'a' value for a flanged section (when the neutral axis falls in the precast section), the approach to analyze a composite beam at ultimate conditions assuming a monolithic section composed of precast beam and cast-in-place slab where the width is transformed based on the compressive strength of the precast beam introduces an error that may be significant. In addition, this approximation results in a higher nominal bending resistance since the compressive strength of the precast beam is used. Therefore, in such a case, the nominal bending resistance was computed by combining the stress in the prestressing steel at nominal bending resistance with the equations of the force and moment equilibrium at ultimate.

To compute the ultimate flexural strength, three cases were considered, one when the neutral axis falls within the slab and the other two when the neutral axis falls within the depth of the precast beam. The same procedures were followed for designs using either the Standard or the LRFD specifications. Figures 5.3 and 5.4 show the neutral axis positions for the three cases considered for the U54 and Type IV beams.

5.3.5.1 Rectangular Beam Section Behavior

If the neutral axis falls within the slab, the nominal moment strength at ultimate conditions will depend on the compressive strength of the slab ($f'_{c\ slab}$) and the effective slab width, as well as the prestressing steel stress. For this case, the reduced nominal moment strength was found as follows.

$$\phi\ M_n = \phi\left[A_{sp} f_{su} d_p \left(1 - \frac{0.6\quad \rho\ f_{su}}{f'_{c\,slab}}\right)\right] \tag{5.8}$$

$$f_{su} = f_{pu}\left[1 - \frac{\gamma}{\beta_1}\rho\frac{f_{pu}}{f'_{c\ slab}}\right] \tag{5.9}$$

$$\rho = \frac{A_{sp}}{b_{effective} d_p} \tag{5.10}$$

where:

M_n	=	Nominal ultimate moment strength
ϕ	=	Flexural strength reduction factor = 1.0 (Standard and LRFD Specifications)
A_{sp}	=	Area of prestressing steel
f_{su}	=	Average stress in prestressing steel at ultimate conditions
d_p	=	Distance from extreme compressive fiber to centroid of the prestressing force
ρ	=	Ratio of prestressing steel
f_{pu}	=	Ultimate stress of prestressing steel
γ	=	Factor for type of prestressing steel (= 0.28) for low relaxation steel (Standard and LRFD Specifications)
β_1	=	Stress block factor(= 0.85 for $f'_c \leq 4.0$ ksi; for $f'_c > 4.0$ ksi, β_1 shall be reduced at a rate of 0.05 for each 1 ksi and shall not be taken less than 0.65)

5.3.5.2 Flanged Section Behavior

If the neutral axis falls within the depth of the precast beam section (Case 1 in Figures 5.3 and 5.4), the traditional transformed compressive slab width (b_{tr}) used for analysis has no meaning at ultimate (large compressive strains). Therefore, the calculations were based on the full effective slab width, as recommended by Nilson (1985). On the other hand, the strain discontinuity at the interface of the CIP slab and the precast section, resulting from prior bending of the non-composite precast section was ignored. However, the stress discontinuity due to differences in concrete compressive strengths at the interface was considered through the use of different equivalent stress blocks at ultimate. According to the neutral axis position, two subcases were considered as described below.

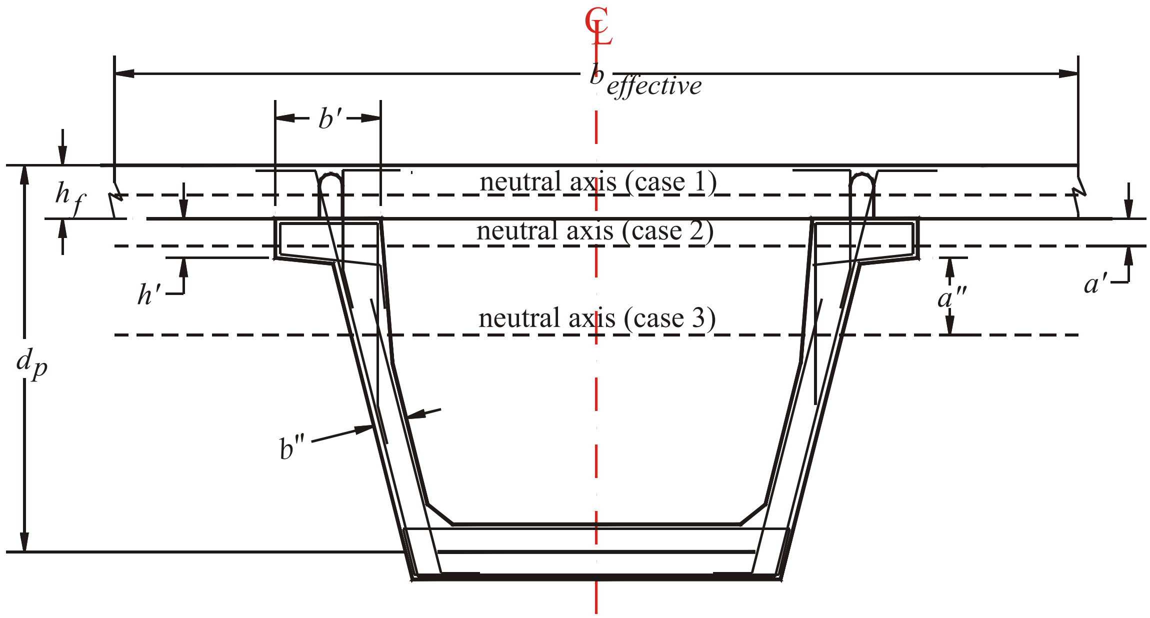


Figure 5.3. Positions of the Neutral Axis in the U54 Beam.

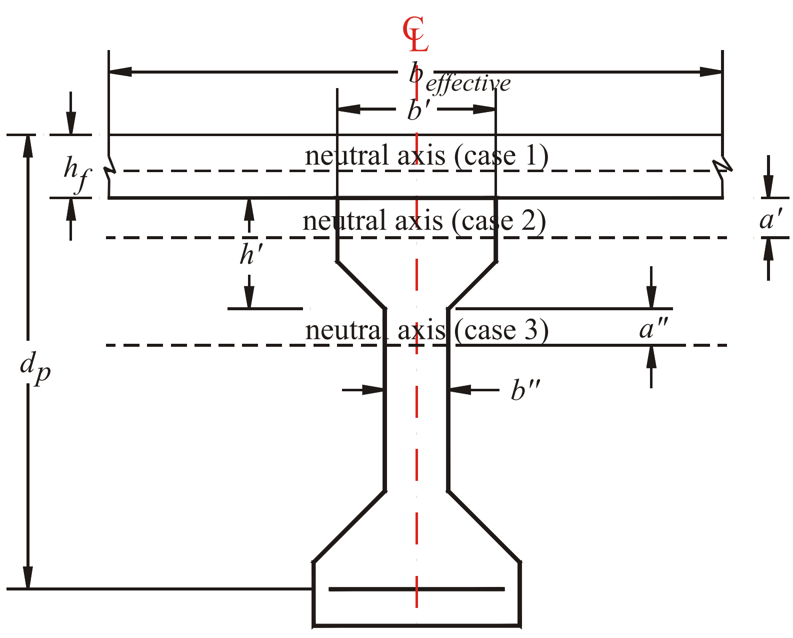


Figure 5.4. Positions of the Neutral Axis in the Type IV Beam.

5.3.5.2.1 Neutral Axis Falls within the Precast Beam Flanges

When the neutral axis depth falls within the precast beam flanges (Case 2 in Figures 5.3 and 5.4), the following equations were used to determine the reduced nominal moment strength at ultimate conditions.

$$\phi M_n = \phi T\left[d_p - (h_f + a')\right] + 0.85 f'_{cslab} h_f \left[\frac{h_f}{2} + \frac{a'}{2}\right] \tag{5.11}$$

$$a' = \frac{A_{sp} f_{su} - 0.85 f'_{cslab} b_{effective} h_f}{0.85 f'_{cp} b'} \tag{5.12}$$

$$T = A_{sp} f_{su} \tag{5.13}$$

where:

M_n	=	Nominal moment strength at ultimate conditions
ϕ	=	Flexural strength reduction factor = 1.0 (Standard and LRFD Specifications)
T	=	Tensile force in the prestressing strands at ultimate conditions
d_p	=	Distance from the extreme compressive fiber to the centroid of the prestressing strands
h_f	=	Depth of the CIP slab (compression flange)
a'	=	Distance between the neutral axis and the compressive face of the precast concrete beam
f'_{cslab}	=	Compressive strength of the slab
f'_{cp}	=	Compressive strength of the precast concrete beam
$b_{effective}$	=	Effective width of the CIP slab (compression flange)
b'	=	Width of the top flanges of the precast beam (2 b' for U54 Beams)
A_{sp}	=	Area of prestressing steel
f_{su}	=	Average stress in prestressing steel at ultimate conditions (Equation 5.9)

5.3.5.2.2 Neutral Axis within Precast Beam Web

When the neutral axis depth falls within the webs of the precast beam (Case 3 in Figures 5.3 and 5.4), the following equations were used to determine the reduced nominal moment at ultimate conditions.

$$\phi M_n = \phi T\left[d_p - \left(h_f + h' + \frac{a''}{2}\right)\right] + 0.85 f'_{cs} b_{effective} h_f \left(\frac{h_f}{2} + h' + \frac{a''}{2}\right) + \left(0.85 f'_{cp} b' h'\right)\left(\frac{h'}{2} + \frac{a''}{2}\right) \qquad (5.14)$$

$$a'' = \frac{A_{sp} f_{su} - 0.85 f'_{cslab} b_{effective} h_f - (0.85 f'_c b' h')}{0.85 f'_{cp} b''} \qquad (5.15)$$

$$T = A_{sp} f_{su} \qquad (5.16)$$

where:

a''	=	Distance between the neutral axis and the bottom face of the precast concrete beam
b''	=	Width of the webs of the precast beam (2 b'' for U54 Beams)
h'	=	Depth of the flange of the precast beam

5.3.5.3 Design Ultimate Moment Strength

The required ultimate moment for design according to the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b) are as follows.

$$M_u = 1.30(M_D + M_{SD} + M_{CSD} + 1.67 M_{L+I}) \qquad \text{AASHTO Standard Specs.} \qquad (5.17)$$

$$M_u = 1.00(1.25 M_D + M_{SD} + M_{CSD} + 1.75 M_{L+I}) \qquad \text{AASHTO LRFD Specs.} \qquad (5.18)$$

The designs must satisfy the following relationship.

$$\phi M_n \geq M_u \qquad (5.19)$$

where:

M_n	=	Nominal moment strength at ultimate conditions
ϕ	=	Flexural strength reduction factor = 1.0 (Standard and LRFD Specifications)
M_u	=	Required ultimate moment for design (factored moment)

5.4 DESIGN PARAMETERS

The selected variables for the overall parametric study are shown in Table 5.3. Table 5.4 shows additional design variables considered in the parametric study.

Table 5.3. Design Parameters.

Variable	Description / Selected Values
Codes	AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b)
Concrete Strength (psi)	6000, 8000, 10000, and 12000 (f'_{ci} is initially set at 0.75 f'_c, but allowed to vary up to f'_c)
Girder Sections	Texas U54 and AASHTO Type IV
Girder Spacing for U54 Beams (ft.)	8.5, 10, 11.5, 14.0, and 16.67
Girder Spacing for Type IV Beams (ft.)	4.25, 5, 5.75, 7.0, 8.5, and 9
Spans (L)	90 ft. to maximum span at 10 ft. intervals
Diameter Strands (inches)	0.5 and 0.6

Table 5.4. Additional Design Variables.

Category	Description	Selected Parameter
Prestressing Strands	Ultimate Strength (f_{pu})	270 ksi – low relaxation
	Jacking Stress Limit (f_{pj})	0.75 f_{pu}
	Yield Strength (f_{py})	0.9 f_{pu}
Concrete-Precast	Unit Weight (w_c)	150 pcf
	Modulus of Elasticity (E_p)	$33\, w_c^{1.5} \sqrt{f'_c}$ (f'_c precast)
Concrete-CIP Slab	Unit Weight (w_c)	150 pcf
	Modulus of Elasticity (E_{cip})	$33\, w_c^{1.5} \sqrt{f'_c}$ (f'_c CIP)
	Specified Compressive Strength (f'_c)	4000 psi
	Modular ratio (n)	E_{cip}/E_p
Other	Relative Humidity	75 %
	Non-composite dead loads	Two interior diaphragms of 3 kips each, accounted as two concentrated dead loads each located at 10 ft. from the beam midspan (only for U54 Beams). Haunch and overlay loads are not used
	Composite dead loads	1/3 of a T501 rail (1/3*0.333 klf = 0.110 klf)

Table 5.4. Continued

Category	Description	Selected Parameter
	Debonding Length in U54 Beams	$L \leq 100$ ft.: the lesser of 0.2 L or 15 ft. 100 ft. < L <120 ft. : 0.15 L $L \geq 120$ ft.: 18 ft.
	Harping in Type IV	An allowable harping pattern will be selected to limit the stresses to the required values.

Both the AASHTO Standard and LRFD Specifications were studied. Concrete strengths from 6000 to 12000 psi were considered because this range is reasonably available and acceptable in bridge design. The AASHTO Type IV and Texas U54 Beams were chosen because they are widely used for long span bridges. The Type IV beam has been the dominant beam since 1986, and the TxDOT beam is an alternative to the Type IV beam. Girder spacings for the U54 beam were based on a reasonable deck width and an integer number of beams up to the maximum spacing. Girder spacings for the Type IV beam were set as one half of the spacing used for the U54 beam for comparison with the U54 beam spacings. The minimum and maximum girder spacings were chosen according to recommendations given in the Texas Bridge Design Manual (TxDOT 2001). The calculations were performed for spans, which were considered relatively long, ranging from 90 ft. to maximum span lengths at 10 ft. intervals. Low relaxation 0.5 and 0.6 in. diameter strands with an ultimate strength of 270 ksi were used. The larger 0.6 in. diameter strands have been tested and are allowed to be used. Their application is typically most beneficial for higher concrete strengths.

The assumed debonding length for the U54 Beams is given in Table 5.4. The harping pattern for the Type IV beams followed the approach used in the PSTRS14 program (TxDOT 1980), where the tendons are held down at a distance of 5 ft., or 0.05 times the span length, whichever is greater, on either side of the beam midspan. Each row of stands that is harped is raised such that successive rows of strands do not cross one another (i.e., the row of strands farthest from the bottom face of the beam at midspan remains as the row farthest from the bottom of the beams at the ends). Strands are raised in pairs for each row. For the parametric study, two strands for all rows were harped.

5.5 CASE STUDIES

Prior to beginning the full parametric study, several case study beams were checked to ensure that the design and analysis approach was consistent with TxDOT's standard practices. Specifically, the results from the spreadsheet program used for this study were compared with the results of the PSTRS14 program used by TxDOT (1980). Since the PSTRS14 program was developed for only the AASHTO Standard Specifications, comparisons were made only for this code.

5.5.1 Two Case Studies for U54 Beams

Several case study bridges for U54 Beams designed under the Standard Specifications were examined. Designs from this study were compared with those from the PSTRS14 program. Table 5.5 shows the design variables for two case studies.

Table 5.5. U54 Case Study Bridges – Design Variables.

Parameters	Case 1 - Exterior	Case 2 - Interior
SPAN LENGTH	124.9 ft.	124.9 ft.
TYPE OF BEAM	U54 beam	U54 beam
GIRDER SPACING	10.9 ft.	8.26 ft.
CODE	Standard Specifications	Standard Specifications
LOADS		
Live Load	HS25*	HS25*
Impact Factor	1.2	1.2
Distribution Factor	0.892	0.9
Non-Composite Loads: Haunch	0.171 klf	0.027 klf
2 Diaphragms	3 kips	3 kips
Composite Loads	0.393 klf	0.197 klf
MATERIALS		
Concrete Strength – CIP Slab	3600 psi	3600 psi
Concrete Strength – Precast beam	To be calculated	To be calculated
Unit weight of Beam and Slab	0.150 kcf	0.150 kcf
Strand Ultimate Strength	270 ksi - low relaxation	270 ksi - low relaxation
Strand Diameter	0.6 in.	0.6 in.
PRESTRESS LOSSES	AASHTO - refined method	AASHTO - refined method
OTHER		
Relative Humidity	75%	75%
Debonded Length	18 ft.	18 ft.
Modular ratio: $n=E_{CIP}/E_{precast}$	1.0	1.0

* Note: HS25 is 25% larger than the HS20 truck loading.

Table 5.6 shows a comparison of designs obtained using the PSTRS14 program with those obtained from this study, based on the Standard Specifications. For all design variables, very small differences (between 0.0 to 1.45 percent) were found. The largest difference was found in the calculation of ϕM_n. This is likely due to the refinements used in this study for calculating M_n. Based on these small differences, designs from this study and those from the PSTRS14 were deemed consistent. The results of the parametric study for U54 Beams are provided in Section 6.

Table 5.6. U54 Case Study Bridges – Comparison of Results.

Design Results		Case 1 - Exterior Beam			Case 2 - Interior Beam		
		TxDOT PS14	**This Study**	**% Diff.**	**TxDOT PS14**	**This Study**	**% Diff.**
Req. Concrete Strength (psi)							
Initial (f'_{ci}):		7648	7649	0.01	6081	6081	0.00
Final (f'_c):		8720	8716	-0.05	7007	7005	0.03
Stresses (psi)							
Release (at ends)	Top	505	503	-0.40	310	308	-0.65
	Bottom	-4589 $F'_{ci} = 7648$	-4589 $f'_{ci} = 7649$	0.00	-3649 $f'_{ci} = 6081$	-3648 $f'_{ci} = 6081$	-0.03
Interm. Stage (at midspan)	Top	-3488 $F'_c = 8720$	-3486 $f'_c = 8716$	-0.06	-2803 $f'_c = 7007$	-2802 $f'_c = 7005$	-0.04
Final Stage (at midspan)	Top	-3904	-3902	-0.05	-3343	-3342	-0.03
	Bottom	483	481	-0.41	435	435	0.03
Number of Strands		70	70	0.00	56	56	0.00
Losses (ksi)							
Initial:		19.617	19.605	-0.06	17.225	17.234	0.05
Final:		56.919	56.892	-0.05	48.572	48.613	0.08
M_u (kip-ft.)		13070	13071	0.01	11590	11593	0.03
ϕM_n (kip-ft.)		16874	17036	0.96	13522	13719	1.46

5.5.2 Two Case Studies for Type IV Beams

Several case study bridges for Type IV beams designed under the Standard Specifications were examined. Designs from this study were compared with those from the PSTRS14 (TxDOT 1980). Table 5.7 shows the design variables for two case studies.

Table 5.7. Type IV Case Study Bridges – Design Variables.

Parameters	Case 1 - Interior Beam	Case 2 - Interior Beam
SPAN LENGTH (L)	125.6 ft.	145.0 ft.
TYPE OF BEAM (S)	Type IV beam	Type IV beam
GIRDER SPACING	8.5 ft.	4.25 ft.
CODE	Standard Specifications	Standard Specifications
LOADS		
Live Load	HS20	HS20
Impact Factor	1.2	1.2
Distribution Factor	0.773	0.386
Non-Composite Loads: Haunch	0 klf	0 klf
Diaphragms	0 kips	0 kips
Composite Loads	0.110 klf	0.110 klf
MATERIALS		
Concrete Strength - CIP Slab	3600 psi	3600 psi
Concrete Strength - Precast Beam	To be calculated	To be calculated
Unit Weight of Beam and Slab	0.150 kcf	0.150 kcf
Strand Ultimate Strength	270 ksi - low relaxation	270 ksi - low relaxation
Strand Diameter	0.5 in.	0.5 in.
PRESTRESS LOSSES	AASHTO - refined method	AASHTO - refined method
OTHER		
Relative Humidity	75%	75%
Number of Draped Strands	32 ft.	12 ft.
Modular Ratio: $n=E_{CIP}/E_{precast}$	1.0	1.0

In the parametric study, the modular ratio was computed as the value corresponding to the ratio between the modulus of elasticity of the CIP slab and the precast beam. However, the comparisons presented in this section use a modular ratio of 1.0, consistent with TxDOT practices.

Table 5.8 shows a comparison of designs obtained using the PSTRS14 program with those obtained from this study, based on the Standard Specifications. For all design variables, very small differences (between 0.0 to 3.99 percent) were found. The only difference that is more than 1.0 percent is in the calculation of M_n. Thus, consistency between designs from this study and those from the PSTRS14 was confirmed. The results of the parametric study for Type IV beams are provided in Section 7.

Table 5.8. Type IV Case Study Bridges – Comparison of Results.

Design Results		Case 1 (L = 125.6 ft., S = 8.5 ft.)			Case 2 (L = 145.0 ft., S = 4.25 ft.)		
		TxDOT PS14	**This Study**	**% Diff.**	**TxDOT PS14**	**This Study**	**% Diff.**
Concrete Strength Req. (psi)							
Initial (f'_{ci}):		7156	7156	0.00	8408	8408	0.00
Final (f'_c):		9767	9767	0.00	6467	6467	0.03
Stresses (psi)							
Release (at ends)	Top	-1254	-1254	0.00	-155	-156	0.65
	Bottom	-4293 f'_{ci} =7156	-4293 f'_{ci} =7156	0.00	-3880 f'_{ci} =6467	-3880 f'_{ci} =6467	0.00
Interm. Stage (at midspan)	Top	-3906 f'_c =9767	-3906 f'_c =9767	0.0	-3363 f'_c =8408	-3363 f'_c =8408	0.00
Final Stage (at midspan)	Top	-4291	-4290	-0.02	-3773	-3773	0.00
	Bottom	573	570	-0.52	432	430	-0.46
Number of Strands		82	82	0.00	60	60	0.00
Losses (ksi)							
Initial:		9.99	9.99	0.00	15.876	15.870	-0.04
Final:		30.31	3030	-0.03	45.710	45.964	-0.04
M_u (kip-ft.)		8551	8552	0.01	6950	6950	0.00
ϕM_n (kip-ft.)		11915	12305	3.27	8829	9181	3.99

6 RESULTS FOR U54 BEAMS

6.1 INTRODUCTION

Researchers conducted a parametric study composed of a number of designs using Texas U54 prestressed concrete bridge girders. The main objective was to investigate the controlling limit states and the impact of varying the concrete compression strength of the precast section, strand diameters, girder spacing, and code requirements. The flexural limit states (service and ultimate) were included in this study. A summary of the design parameters is given in Table 6.1, and additional details are provided in Section 5.5.

Table 6.1. Summary of Design Parameters.

Parameter	Description and Selected Values
Codes	AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b)
Concrete Strength (psi) (f'_c)	6000, 8000, 10000, and 12000 (f'_{ci} was initially set at 0.75 f'_c, but allowed to vary up to f'_c)
Girder Spacing (ft.)	8.5, 10, 11.5, 14.0, and 16.67
Spans	90 ft. to maximum span at 10 ft. intervals

For the parametric study, the span lengths were varied from 90 ft. to the maximum span possible for a given set of parameters, using 10 ft. increments. For each of these spans, the most economical design (fewest number of strands) was determined and the corresponding controlling limit state was identified. In the discussion of results, the span lengths are labeled as "shorter spans," "longer spans," and "maximum spans." A shorter span is generally in the range of 90 to 100 ft. long. A longer span is a span that is greater than 100 ft. up to, but not including, the maximum span length. The maximum span length is the length beyond which a flexural limit state would be exceeded, such that for the particular set of parameters the span has been maximized. For every case studied, key design information is available in tables provided in Appendix C. Based on these results, the following sections summarize the findings, with a primary focus on the maximum spans.

6.2 DESCRIPTION OF CONTROLLING LIMIT STATES

A controlling limit state is defined for this study as the flexural design limit state that dictates the required number of strands for a given geometry and demand. In the case of establishing the maximum span length, the controlling limit state is defined as the limit state that would be exceeded if the span was increased. Limit states include satisfying the allowable stresses and required ultimate flexural strength, both at the maximum moment section along the span and at the beam ends. The required number of strands (prestressing force) is determined to ensure that the allowable stresses are not exceeded as the beam is loaded from the initial to the final service stage. In addition, the ultimate flexural strength is checked. The required number of strands is computed using a systematic approach that is based on attaining actual stresses as near as possible to the corresponding allowable stresses for the considered load stages to achieve the most economical design (see Section 5.4).

The number of strands, and consequently span lengths, are primarily controlled by one of the four allowable stresses: compressive and tensile stresses at the beam ends upon release of the prestressing strands, compressive sustained load stresses, and tensile service load stress; or by the required flexural strength at ultimate conditions. The compressive service load stress and the stresses at midspan at release were also considered in the design, but were not critical. Combinations of the controlling limit states were also considered for the cases where temporary allowable stresses at the beam ends or eccentricity limitations initially control the number of strands that may be used, followed by exceeding the allowable stresses for the sustained or service load conditions. According to the limits above, Table 6.2 identifies flexural limit states that control the required number of strands for maximum span lengths for the U54 girders.

Table 6.2. Controlling Flexural Limit States for U54 Girders.

Controlling Limit State	Description
Flexural Strength	Required flexural strength at ultimate.
Flexural Strength**	The number of strands is initially limited by the concrete tensile stress at the beam ends at release, followed by the required flexural strength at ultimate conditions.
f(t) Total Load	The number of strands is controlled by the concrete tensile stress at midspan at the final stage due to total loads (including live loads).
f(c) Total Dead Load	The number of strands is controlled by the concrete compressive stress at midspan at the intermediate stage due to total dead loads (not including live loads).
f(t) Total Load ^{99}	The number of strands is controlled by the concrete tensile stress at midspan at the final stage due to total loads. Unlike the same limit state defined above, this occurs when the maximum number of strand positions is used (99 for U54 Beams).
f(t) Total Load*	The number of strands is initially limited by the concrete compressive stress at the beam ends at release, followed by the concrete tensile stress at midspan at the final stage.
f(c) Total Dead Load*	The number of strands is initially limited by the concrete compressive stress at the beam ends at release, followed by the concrete compressive stress at midspan at the intermediate stage due to sustained loads.
f(t) Total Load**	The number of strands is initially limited by the concrete tensile stress at the beam ends at release, followed by the concrete tensile stress at midspan at the final stage due to total loads.
f(c) Total Dead Load**	The number of strands is initially limited by the concrete tensile stress at the beam ends at release, followed by the concrete compressive stress at midspan at the intermediate stage due to sustained loads.
f(t) T L / f(c) T D L	The number of strands is initially limited by the concrete compressive stress at midspan at the intermediate stage due to sustained loads, followed by the concrete tensile stress at midspan at the final stage due to total loads.
(f(t) T L & f(c) T D L)**	The number of strands is initially limited by the concrete tensile stress at the beam ends at release followed by the concrete tensile and compressive stress occurring simultaneously.
f(c) Total Dead Load &**	The number of strands is simultaneously limited by the concrete tensile stress at the beam ends at release and the concrete compressive stress at midspan at the intermediate stage due to sustained loads.

6.3 CONTROLLING LIMIT STATES FOR AASHTO STANDARD AND LRFD SPECIFICATIONS

6.3.1 AASHTO Standard Specifications

Tables C.1 through C.10 of Appendix C provide controlling limit states for spans from 90 ft. to maximum span lengths at 10 ft. intervals for different concrete classes and girder spacings of U54 Beams designed using the AASHTO Standard Specifications (2002a). Tables 6.3 through 6.4 show the controlling limit states for the maximum span lengths, together with required number of strands and concrete release strengths, for different concrete classes and girder spacings. Both 0.5 and 0.6 in. diameter strands were considered.

The following trends were observed for these designs. For shorter spans (90 ft. and, in some cases, 100 ft.), the number of strands required is controlled by the required flexural strength. In some cases, it was necessary to increase the number of strands to provide the required flexural strength.

The number of strands required for longer spans, except the maximum span lengths, is controlled by the concrete tensile stress at midspan at the final load stage. Maximum span lengths are controlled by concrete compressive stresses due to total dead loads (not including live loads), except when additional prestressing strands cannot be fit in the U54 beam section (the number of strand positions available in a U54 cross section is 99). In this case, the maximum span lengths are controlled by the concrete tensile stresses. Stresses at the ends during transfer do not control the number of strands in any case when allowable stress limits at release were taken as $0.6\,f'_{ci}$ for compression and $7.5\sqrt{f'_{ci}}$ for tension, where f'_{ci} is in psi units (see Section 5.4).

Table 6.3. Summary of Controlling Limit States and Maximum Spans (AASHTO Standard Specifications, Strand Diameter = 0.5 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	4500	8.5	114.1	57	f(c) Total Dead Load
	4500	10.0	110.1	55	f(c) Total Dead Load
	4500	11.5	106.9	58	f(c) Total Dead Load
	4925	14.0	102.2	63	f(c) Total Dead Load
	5296	16.6	97.8	67	f(c) Total Dead Load
8000	6000	8.5	130.0	80	f(c) Total Dead Load
	6000	10.0	125.3	75	f(c) Total Dead Load
	6000	11.5	121.7	80	f(c) Total Dead Load
	6456	14.0	115.9	85	f(c) Total Dead Load
	6942	16.6	110.2	91	f(c) Total Dead Load
10000	7500	8.5	140.8	96	f(c) Total Dead Load
	7500	10.0	136.5	95	f(c) Total Dead Load
	7500	11.5	130.9	99	f(t) Total Load [99]
	7500	14.0	121.5	99	f(t) Total Load [99]
	7500	16.6	113.2	99	f(t) Total Load [99]
12000	9000	8.5	142.8	99	f(t) Total Load [99]
	9000	10.0	138.8	99	f(t) Total Load [99]
	9000	11.5	132.2	99	f(t) Total Load [99]
	9000	14.0	122.7	99	f(t) Total Load [99]
	9000	16.6	114.5	99	f(t) Total Load [99]

Table 6.4. Summary of Controlling Limit States and Maximum Spans (AASHTO Standard Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	4500	8.5	115.1	40	f(c) Total Dead Load
	4500	10.0	111.2	40	f(c) Total Dead Load
	4638	11.5	108.1	42	f(c) Total Dead Load
	5072	14.0	103.5	45	f(c) Total Dead Load
	5609	16.6	99.4	49	f(c) Total Dead Load
8000	6000	8.5	131.8	55	f(c) Total Dead Load
	6000	10.0	127.3	55	f(c) Total Dead Load
	6205	11.5	123.5	57	f(c) Total Dead Load
	6831	14.0	117.9	62	f(c) Total Dead Load
	7362	16.6	112.7	66	f(c) Total Dead Load
10000	7608	8.5	145.5	71	f(c) Total Dead Load
	7531	10.0	140.5	70	f(c) Total Dead Load
	8008	11.5	136.2	74	f(c) Total Dead Load
	8830	14.0	130.0	81	f(c) Total Dead Load
	9579	16.6	123.7	88	f(c) Total Dead Load
12000	9432	8.5	157.0	89	f(c) Total Dead Load
	9375	10.0	151.8	88	f(c) Total Dead Load
	9828	11.5	146.1	93	f(c) Total Dead Load
	10283	14.0	136.9	99	f(t) Total Load [99]
	9000	16.6	127.5	99	f(t) Total Load [99]

6.3.2 AASHTO LRFD Specifications

Tables C.11 through C.20 of Appendix C provide controlling limit states for spans from 90 ft. to maximum span lengths at 10 ft. intervals for different concrete strengths and spacing of U54 girders designed under the AASHTO LRFD Specifications (2002). Tables 6.5 through 6.6 show the controlling limit states for maximum span lengths, together with the required number of strands and concrete release strengths, for different concrete classes and girder spacings. Both 0.5 and 0.6 in. diameter strands were considered.

The following trends were observed for designs based on the AASHTO LRFD Specifications. Like designs under the Standard Specifications, the number of strands required for shorter spans (90 ft. and, in some cases, 100 ft.) is controlled by the required flexural strength at ultimate conditions. In some cases, it was necessary to increase the number of strands to provide the required flexural strength.

The number of strands required for longer spans, except the maximum span lengths, is controlled by the concrete tensile stress under service loads at midspan. Maximum span lengths are controlled by concrete compressive stresses due to total dead loads (not including live loads), except when additional prestressing strands cannot be fit in the U54 beam. In this case, maximum spans are controlled by the concrete tensile stress at service loads and at midspan. Other exceptions were noted for wider girder spacings where maximum spans are limited because the number of strands that could be used was controlled by the compressive or tensile stress at the beam ends at release.

The concrete strength at release (f'_{ci}) is critical for LRFD designs with the wider girder spacings (14 and 16.6 ft.) and concrete strengths up to 10000 psi. The stress limits at release were taken as $0.6\,f'_{ci}$ for compression and $6.96\sqrt{f'_{ci}}$ for tension, where f'_{ci} is in psi units (see Section 5.4).

Table 6.5. Summary of Controlling Limit States and Maximum Spans (AASHTO LRFD Specifications, Strand Diameter = 0.5 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	4500	8.5	119.6	59	f(c) Total Dead Load
	4547	10.0	115.7	61	f(c) Total Dead Load
	4611	11.5	112.2	63	f(c) Total Dead Load
	5973	14.0	106.3	77	f(t) Total Load*
	5994	16.6	98.9	76	f(t) Total Load*
8000	6018	8.5	136.3	82	f(t) Total Dead Load
	6276	10.0	131.8	85	f(c) Total Dead Load
	6448	11.5	127.6	87	f(c) Total Dead Load
	7228	14.0	115.0	99	f(t) Total Load[99]
	7347	16.6	107.6	99	f(t) Total Load[99]
10000	7500	8.5	144.6	99	f(t) Total Load[99]
	7500	10.0	138.3	99	f(t) Total Load[99]
	7500	11.5	132.7	99	f(t) Total Load[99]
	7500	14.0	115.9	99	f(t) Total Load[99]
	7500	16.6	108.3	99	f(t) Total Load[99]
12000	9000	8.5	145.9	99	f(t) Total Load[99]
	9000	10.0	139.5	99	f(t) Total Load[99]
	9000	11.5	133.9	99	f(t) Total Load[99]
	9000	14.0	116.9	99	f(t) Total Load[99]
	9000	16.6	109.3	99	f(t) Total Load[99]

Table 6.6. Summary of Controlling Limit States and Maximum Spans (AASHTO LRFD Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	4500	8.5	120.9	43	f(c) Total Dead Load
	4732	10.0	117.1	44	f(c) Total Dead Load
	4909	11.5	113.6	45	f(c) Total Dead Load
	5977	14.0	106.3	53	f(t) Total Load*
	5932	16.6	95.8	48	f(t) Total Load**
8000	6301	8.5	138.3	59	f(c) Total Dead Load
	6557	10.0	133.8	61	f(c) Total Dead Load
	6699	11.5	129.6	62	f(c) Total Dead Load
	7902	14.0	118.6	72	f(t) Total Load*
	7965	16.6	101.9	55	f(t) Total Load**
10000	8195	8.5	152.9	77	f(c) Total Dead Load
	8556	10.0	147.9	80	f(c) Total Dead Load
	8902	11.5	143.2	83	f(c) Total Dead Load
	9976	14.0	127.9	94	f(t) Total Load*
	9777	16.6	110.1	68	f(t) Total Load**
12000	9937	8.5	162.8	97	f(t) T L / f(c) T D L
	10093	10.0	155.9	99	f(t) Total Load[99]
	10135	11.5	149.5	99	f(t) Total Load[99]
	12277	14.0	129.5	99	f(t) Total Load[99]
	10343	16.6	120.9	99	f(t) Total Load[99]

The significant load demands of LRFD designs using 14 and 16.6 ft. girder spacings require a large number of prestressing strands for service conditions. This corresponds to high initial prestressing forces at the beam ends. The higher the initial prestressing forces, the greater the required initial concrete strengths. Consequently, the initial stresses control because they become even more critical than the final stresses. In this case, there is a need for a high early concrete strength because the optimal time prior to transfer for production is approximately 12 to 24 hours. The strength gain after release is often not as critical in these cases.

For U54 Beams with wider girder spacings (14 and 16.6 ft.) and using 0.5 diameter strands, the allowable compressive stress at the beam ends at release controls the number of strands used for the maximum span lengths for the lowest concrete strength (6000 psi). When the concrete strength is greater than 6000 psi, the maximum span lengths are controlled by the maximum number of strands that the U54 beam can accommodate (99).

For U54 Beams with wider girder spacings (14 and 16.6 ft.) using 0.6 in. diameter strands, the allowable compressive and tensile stresses at the beam ends during transfer control the number of strands that can be used for maximum span lengths for all concrete strengths, except 12000 psi. More specifically, the allowable compressive stress during release controls when the girder spacing of 14 ft. is used with concrete strengths up to 10000 psi. The allowable tensile stress during release controls when the girder spacing of 16.6 ft. is used with concrete strengths up to 10000 psi. When wider girder spacings are used with an f'_c of 12000 psi, the maximum spans are controlled by the maximum number of strands that the U54 beam can accommodate (99).

6.4 STRAND DIAMETER AND CONCRETE STRENGTH

6.4.1 General

One purpose of the parametric study was to determine the increase in span length possible through the use of different concrete classes. However, the effective use of concrete depends on the diameter of the strands; therefore, the impact of strand diameter was also studied. Figures

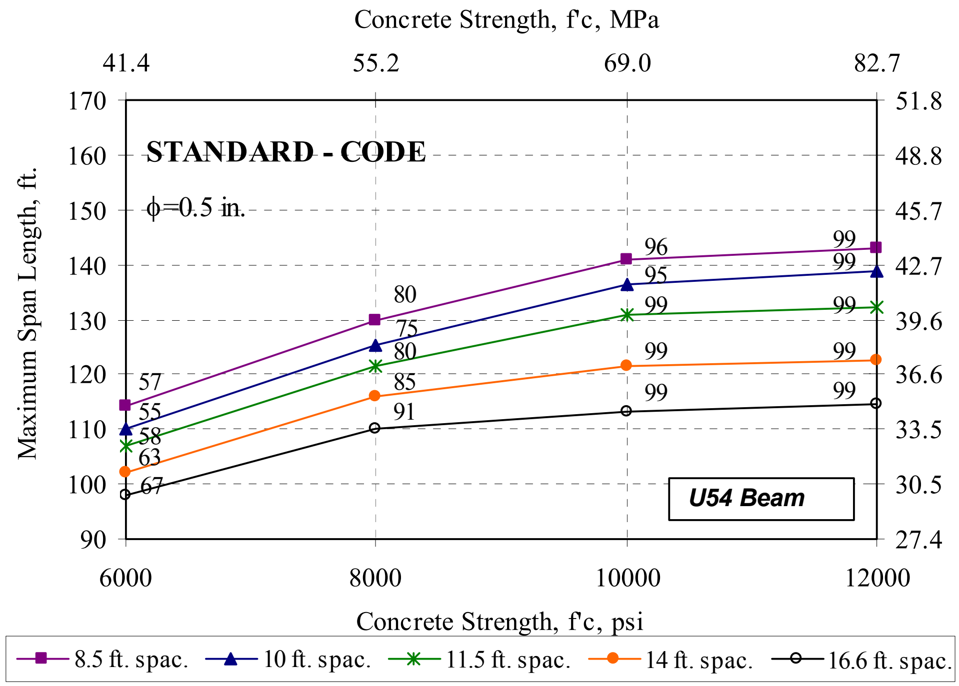
6.1 and 6.2 show the trends for maximum span lengths versus various concrete strengths for each girder spacing considered. These graphs correspond to strand diameters of 0.5 and 0.6 in., following both the AASHTO Standard and LRFD Specifications. These graphs help to describe how the strand diameter impacts the effective use of concrete strength and, consequently, the maximum span lengths that can be attained. These figures show that for the 0.5 in. diameter strands, the maximum achievable span nearly levels off beyond a certain concrete strength. This leveling off occurs when additional prestressing strands cannot be added to the U54 beam cross-section due to space limitations (the maximum number of strand locations in a U54 beam is 99). In such cases, the girder section cannot efficiently use higher concrete strengths.

6.4.2 Trends Observed for AASHTO Standard Specifications

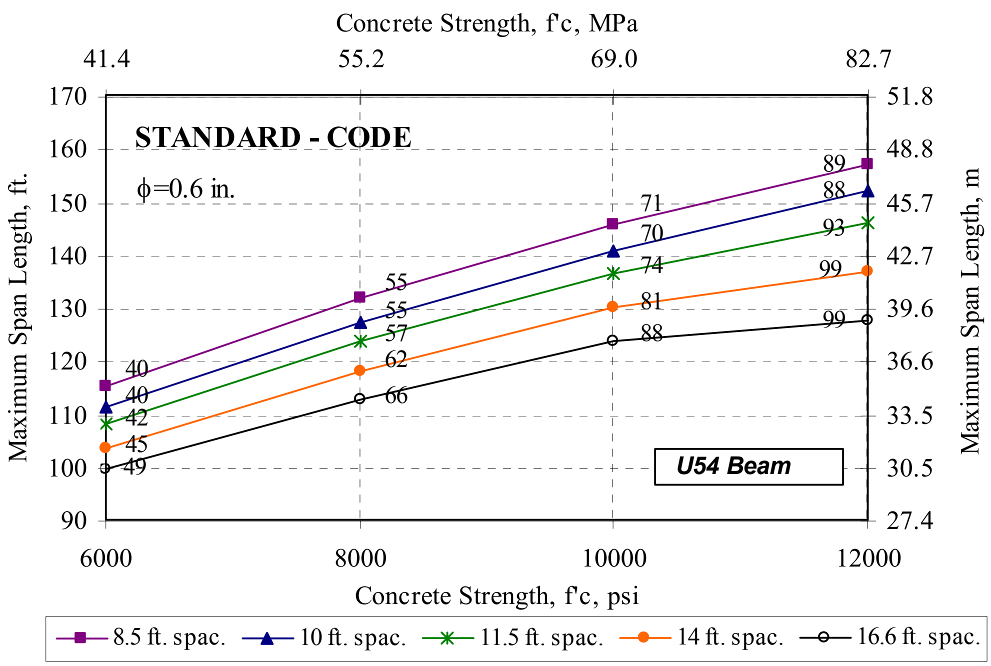
Figure 6.1a shows that the U54 beam with 0.5 in. diameter strands designed using the AASHTO Standard Specifications can fully utilize concrete strengths up to 10000 psi. The maximum span lengths nearly level off at this strength. Figure 6.1b shows that the U54 beam with 0.6 in. diameter strands can fully utilize concrete compressive strengths up to 12000 psi and beyond in some cases (12000 psi was the maximum strength considered).

6.4.3 Trends Observed for AASHTO LRFD Specifications

For the LRFD designs, two different trends were found for each diameter strand considered. These trends are a function of the girder spacings. Figure 6.2a shows that the U54 beam with girder spacings less than 11.5 ft. using 0.5 in. diameter strands can effectively use concrete compressive strengths up to 10000 psi. Above this f'_c, the maximum span lengths nearly level off. For girder spacing more than 11.5 ft., the U54 beam can effectively use concrete strengths only up to 8000 psi. In this case, the more stringent distribution factors for wider girder spacings result in larger live load demands. The span lengths are then controlled by the maximum number of strands (no more than 99) that the U54 beam can accommodate.

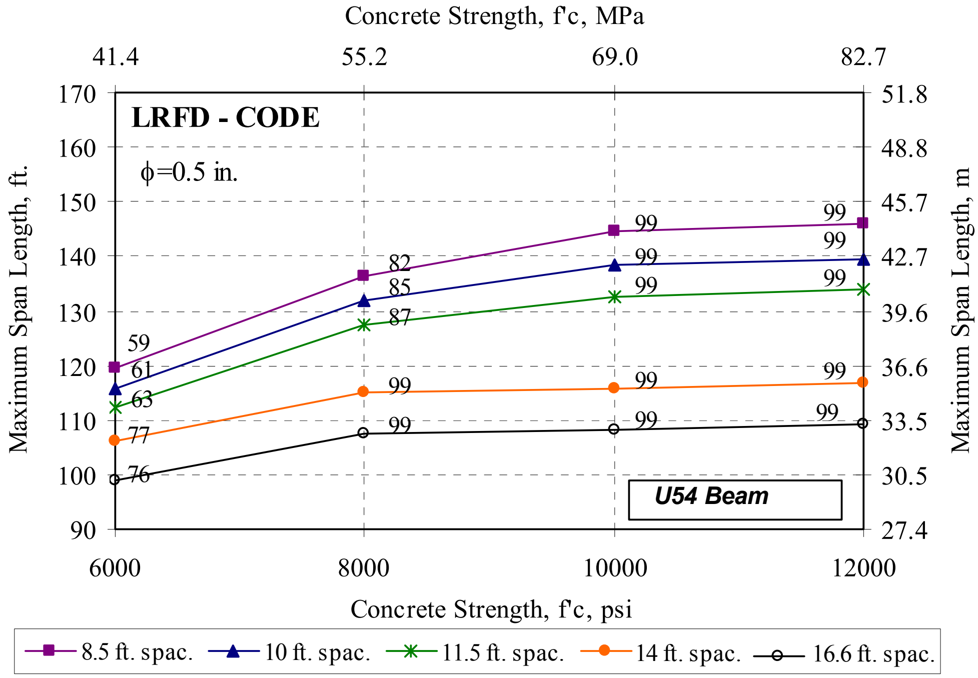


(a) Strand Diameter = 0.5 in.

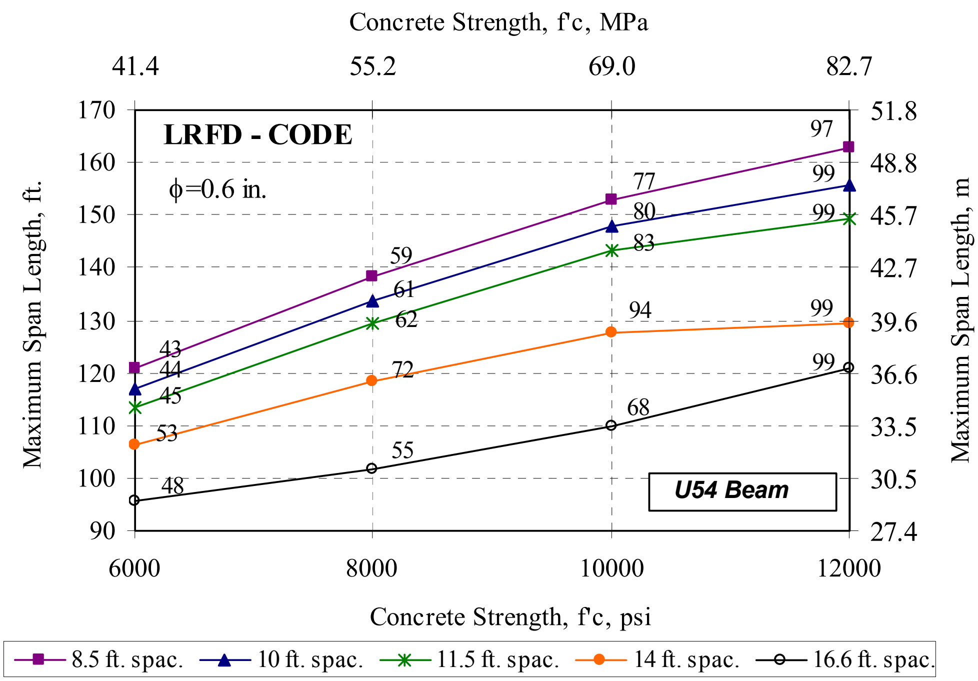


(b) Strand Diameter = 0.6 in.

Figure 6.1. AASHTO Standard Specifications – Maximum Span Length versus Concrete Strength for U54 Girders.



(a) Strand Diameter = 0.5 in.



(b) Strand Diameter = 0.6 in.

Figure 6.2. AASHTO LRFD Specifications – Maximum Span Length versus Concrete Strength for U54 Girders.

Fig. 6.2b shows that the U54 beam using 0.6 in. diameter strands with girder spacing less than 11.5 ft. can fully utilize concrete compressive strengths up to 12000 psi (maximum strength considered). For girder spacing more than 11.5 ft., the U54 beam can also use concrete strengths up to 12000 psi. However, in this case, the stringent loading demands for wider spacings cause the span lengths to be highly reduced because the required prestressing force leads to high stresses at transfer. The initial stresses at transfer primarily control the maximum achievable span lengths due to limits on the number of strands. Table 6.7 summarizes these trends.

Table 6.7. Effective Concrete Strength (U54 Beams).

Strand Diameter (in.)	**Girder Spacing** (ft.)	**Effective Concrete Strength at Maximum Span Length** (psi)	
		Standard	**LRFD**
0.5	$S \leq 11.5$	10000	10000
	$S > 11.5$	10000	8000
0.6	$S \leq 11.5$	12000	12000
	$S > 11.5$	12000	12000

6.4.4 Impact of Strand Diameter on Maximum Spans

Larger prestressing forces are possible to fully utilize HSC when 0.6 in. diameter strands are used. A comparison of achievable maximum spans for 0.5 and 0.6 in. strand diameters is shown in Tables 6.8 and 6.9 for the Standard and LRFD Specifications, respectively. In general, the results show an increase in maximum spans for a given f'_c and girder spacing when 0.6 in. diameter strands are used instead of 0.5 in. Percentage increases in maximum span up to about 12 percent were found when using 0.6 in. diameter strands for both specifications. As an exception, Table 6.9 shows percentage decreases up to 5.3 percent for two maximum spans designed for the widest girder spacing (16.6 ft.) and concrete strengths less than 8000 psi under the LRFD Specifications.

Table 6.8. Maximum Spans for 0.5 in. and 0.6 in. Diameter Strands (AASHTO Standard Specifications).

f'_c (psi)	Girder Spacing (ft.)	Maximum Span Length (ft.)		Difference ft. (%)
		Strand Diameter = 0.5 in.	Strand Diameter = 0.6 in.	
6000	8.5	114.1	115.1	0.9 (0.8)
	10.0	110.1	111.2	1.1 (1.0)
	11.5	106.9	108.1	1.2 (1.1)
	14.0	102.2	103.5	1.3 (1.3)
	16.6	97.8	99.4	1.6 (1.6)
8000	8.5	130.0	131.8	1.8 (1.4)
	10.0	125.3	127.3	2.1 (1.6)
	11.5	121.7	123.5	1.8 (1.5)
	14.0	115.9	117.9	2.0 (1.7)
	16.6	110.2	112.7	2.6 (2.3)
10000	8.5	140.8	145.5	4.7 (3.3)
	10.0	136.5	140.5	3.9 (2.9)
	11.5	130.9	136.2	5.3 (4.1)
	14.0	121.5	130.0	8.5 (7.0)
	16.6	113.2	123.7	10.5 (9.3)
12000	8.5	142.8	157.0	14.2 (9.9)
	10.0	138.8	151.8	13.0 (9.4)
	11.5	132.2	146.1	13.9 (10.5)
	14.0	122.7	136.9	14.2 (11.5)
	16.6	114.5	127.5	13.0 (11.4)

Table 6.9. Maximum Spans for 0.5 in. and 0.6 in. Diameter Strands (AASHTO LRFD Specifications).

f'_c (psi)	Girder Spacing (ft.)	Max. Span Length (ft.)		Difference ft. (%)
		Strand Diameter = 0.5 in.	Strand Diameter = 0.6 in.	
6000	8.5	119.6	120.9	1.4 (1.1)
	10.0	115.7	117.1	1.3 (1.2)
	11.5	112.2	113.6	1.3 (1.2)
	14.0	106.3	106.3	0.0 (0.0)
	16.6	98.9	95.8	-3.0 (-3.1)
8000	8.5	136.3	138.3	2.0 (1.5)
	10.0	131.8	133.8	2.0 (1.5)
	11.5	127.6	129.6	2.1 (1.6)
	14.0	115.0	118.6	3.6 (3.1)
	16.6	107.6	101.9	-5.6 (-5.3)
10000	8.5	144.6	152.9	8.3 (5.8)
	10.0	138.3	147.9	9.6 (7.0)
	11.5	132.7	143.2	10.5 (7.9)
	14.0	115.8	127.9	12.1 (10.4)
	16.6	108.3	110.1	1.8 (1.6)
12000	8.5	145.9	162.8	16.9 (11.6)
	10.0	139.5	155.9	16.4 (11.7)
	11.5	133.9	149.5	15.6 (11.7)
	14.0	116.9	129.5	12.6 (10.8)
	16.6	109.3	120.9	11.6 (10.6)

6.4.5 Effect of Increasing Concrete Compressive Strengths

Figures 6.3 and 6.4 plot maximum span length versus girder spacing for various concrete compressive strengths. The numerical values adjacent to each data point indicate the required number of strands for the corresponding set of design parameters. These graphs show the benefits of higher concrete compressive strengths in terms of increased maximum spans. Table 6.10 shows the percentage increase in maximum spans when raising f'_c from 6000 psi to the maximum effective strength. Average increases in the maximum span length are differentiated for a girder spacing less than or equal to 11.5 ft. and for more than 11.5 ft.

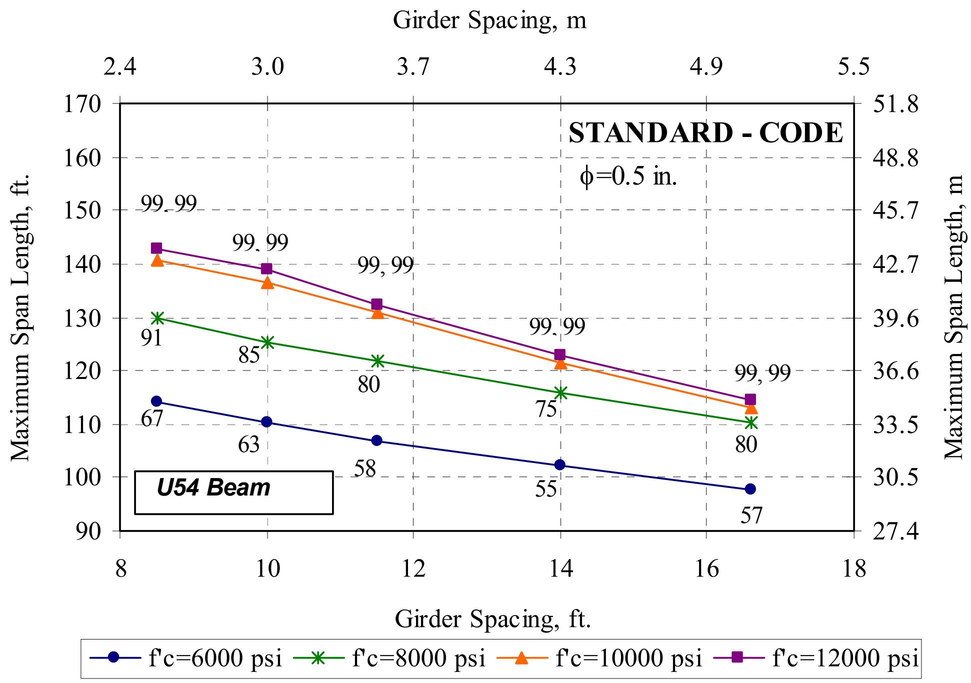
Table 6.10. Impact of Increasing Concrete Compressive Strengths.

Strand Diameter (in.)	**Girder Spacing** (ft.)	**Average Increase in Max. Span Length,** ft. (%)		**Effective Range of Concrete Strength** (psi)	
		Standard	**LRFD**	**Standard**	**LRFD**
0.5	S ≤ 11.5	25 (23)	23 (20)	6000 – 10000	6000 – 10000
	S > 11.5	17 (17)	10 (9)	6000 – 10000	6000 – 8000
0.6	S ≤ 11.5	40 (36)	39 (33)	6000 – 12000	6000 – 12000
	S > 11.5	31 (30)	24 (24)	6000 – 12000	6000 – 12000

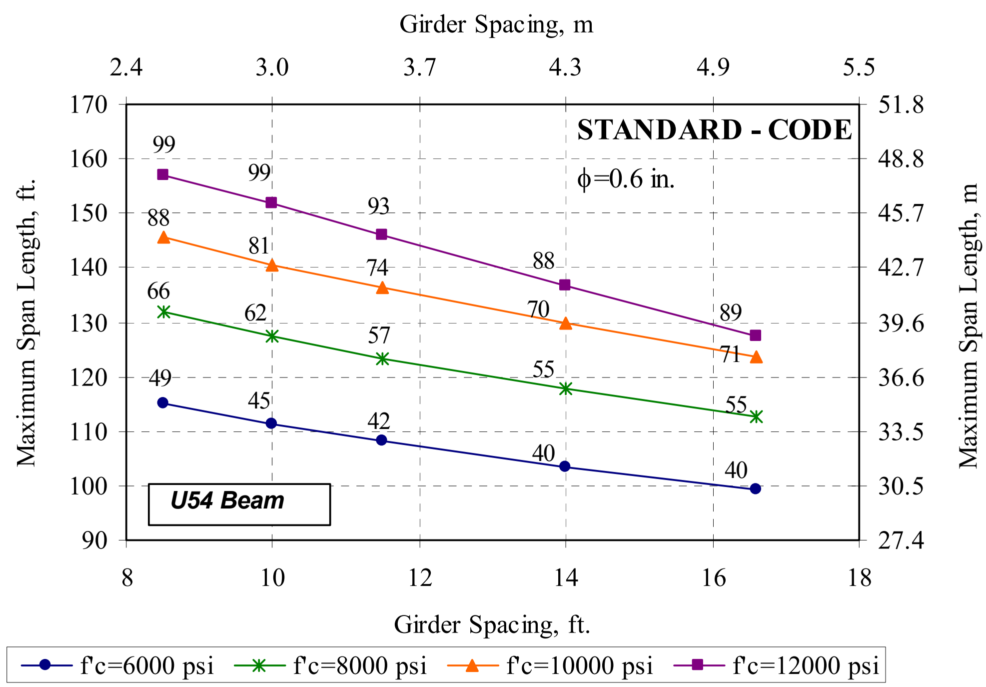
6.5 COMPARISON OF AASHTO STANDARD AND LRFD SPECIFICATIONS

6.5.1 Comparison of Controlling Limit States

Trends for the controlling limit states for shorter, longer, and maximum span lengths are summarized below. Comparisons of limit states that control the maximum spans are shown in Tables 6.11 and 6.12. Most of the shorter spans are controlled by the required nominal flexural strength, while longer span lengths are controlled by the tensile stresses due to the total load at midspan for both specifications.

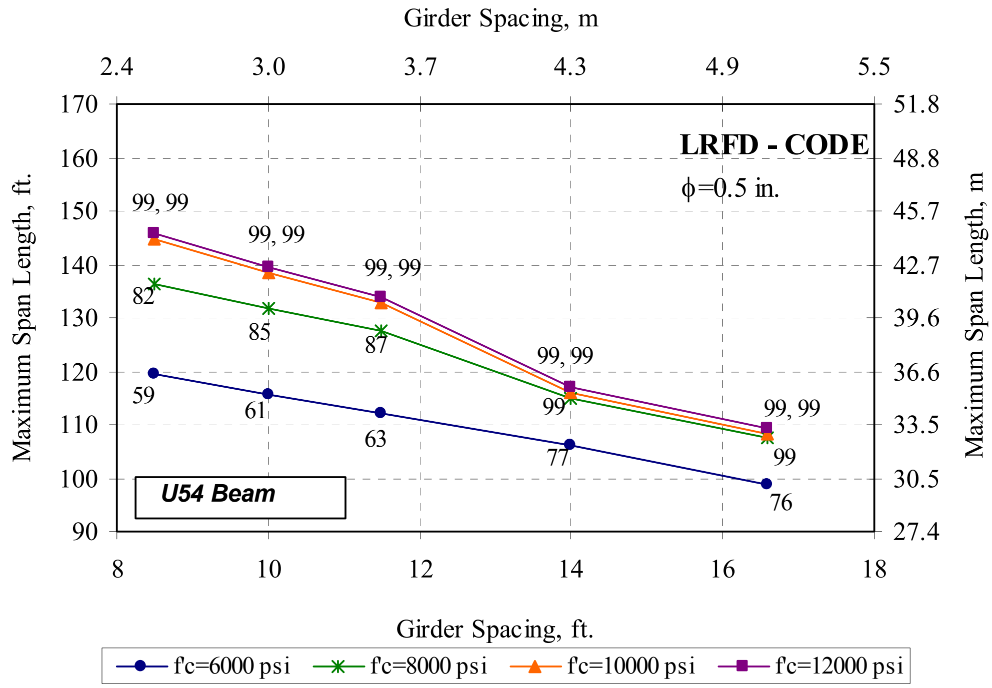


(a) Strand Diameter = 0.5 in.

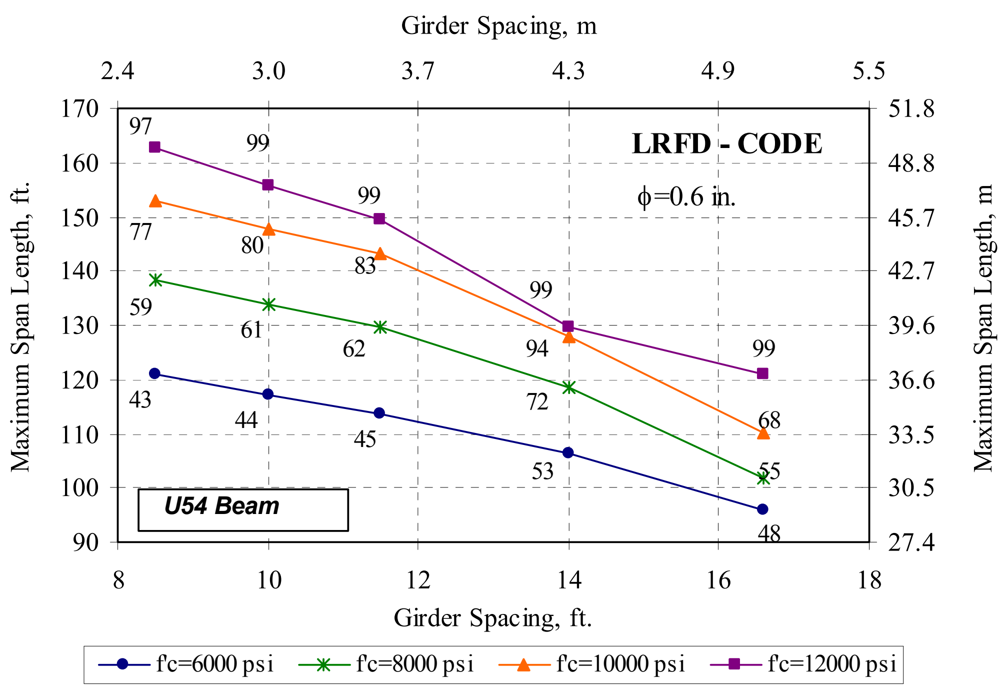


(b) Strand Diameter = 0.6 in.

Figure 6.3. AASHTO Standard Specifications – Maximum Span Length versus Girder Spacing for U54 Girders.



(a) Strand Diameter = 0.5 in.



(b) Strand Diameter = 0.6 in.

Figure 6.4. AASHTO LRFD Specifications – Maximum Span Length versus Girder Spacing for U54 Girders.

Table 6.11. Comparison of Limit States that Control Maximum Span for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.5 in.).

f'c (psi)	**Girder Spacing** (ft.)	**Controlling Limit State**	
		Standard	**LRFD**
6000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(c) Total Load*
	16.6	f(c) Total Dead Load	f(t) Total Load*
8000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(t) Total Load [99]
	16.6	f(c) Total Dead Load	f(t) Total Load [99]
10000	8.5	f(c) Total Dead Load	f(t) Total Load [99]
	10.0	f(c) Total Dead Load	f(t) Total Load [99]
	11.5	f(t) Total Load [99]	f(t) Total Load [99]
	14.0	f(t) Total Load [99]	f(t) Total Load [99]
	16.6	f(t) Total Load [99]	f(t) Total Load [99]
12000	8.5	f(t) Total Load [99]	f(t) Total Load [99]
	10.0	f(t) Total Load [99]	f(t) Total Load [99]
	11.5	f(t) Total Load [99]	f(t) Total Load [99]
	14.0	f(t) Total Load [99]	f(t) Total Load [99]
	16.6	f(t) Total Load [99]	f(t) Total Load [99]

Table 6.12. Comparison of Limit States that Control Maximum Span for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in.).

f'c (psi)	**Girder Spacing** (ft.)	**Controlling Limit State**	
		Standard	**LRFD**
6000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(t) Total Load*
	16.6	f(c) Total Dead Load	f(t) Total Load**
8000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(t) Total Load*
	16.6	f(c) Total Dead Load	f(t) Total Load**
10000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(t) Total Load*
	16.6	f(c) Total Dead Load	f(t) Total Load**
12000	8.5	f(c) Total Dead Load	f(t) T L / f(c) T D L
	10.0	f(c) Total Dead Load	f(t) Total Load [99]
	11.5	f(c) Total Dead Load	f(t) Total Load [99]
	14.0	f(t) Total Load [99]	f(t) Total Load [99]
	16.6	f(t) Total Load [99]	f(t) Total Load [99]

The controlling limit states for the maximum span lengths are different for the Standard and LRFD Specifications. In general, maximum span lengths are controlled by the compressive stresses due to the total dead loads (sustained loads) whether they are designed under the Standard or LRFD Specifications. The exception is when the allowable tensile limit stress under total loads would be exceeded because no additional prestressing strands can be accommodated in the U54 beam section or because the stresses at the beam ends during transfer initially limit the number of strands. Designs using concrete strengths beyond the effective concrete strength do not provide a significant gain in length because no additional strands can be fit into the U54 beam section. Therefore, designs using strengths beyond the effective strengths are not taken into account in establishing these trends (see Table 6.7).

6.5.2 Comparison of Maximum Span Lengths

6.5.2.1 General

Tables 6.13 and 6.14 show the difference in the maximum span lengths for the LRFD and Standard Specifications, for 0.5 and 0.6 in. diameter strands, respectively. The required number of strands is also provided for each design. The difference in the maximum span length for the LRFD designs is expressed as a percentage change relative to the Standard designs. Table 6.15 provides values for the largest differences in the maximum span lengths. Results are shown only for f'_c values up to those strengths that work effectively with the U54 Beams under both codes (where the span lengths are consistently increasing under both specifications). Increases in maximum spans were also differentiated for girder spacings that are within the range of applicability of the LRFD distribution factor equations (less than 11.5 ft.) and for girder spacings greater than 11.5 ft. where the lever rule was used.

Table 6.13. Comparison of Maximum Span Lengths for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.5 in.).

f'_c (psi)	Girder Spacing (ft.)	Standard		LRFD		Span Difference ft. (%)
		Max. Span (ft.)	**No. Strands**	**Max. Span** (ft.)	**No. Strands**	
6000	8.5	114.1	57	119.6	59	5.4 (4.8)
	10.0	110.1	55	115.7	61	5.6 (5.1)
	11.5	106.9	58	112.2	63	5.3 (5.0)
	14.0	102.2	63	106.3	77	4.1 (4.0)
	16.6	97.8	67	98.9	76	1.0 (1.0)
8000	8.5	130.0	80	136.3	82	6.3 (4.9)
	10.0	125.3	75	131.8	85	6.5 (5.2)
	11.5	121.7	80	127.6	87	5.9 (4.8)
	14.0	115.9	85	115.0	99	-0.9 (-0.8)
	16.6	110.2	91	107.6	99	-2.6 (-2.4)
10000	8.5	140.8	96	144.6	99	3.8 (2.7)
	10.0	136.5	95	138.3	99	1.8 (1.3)
	11.5	130.9	99	132.7	99	1.8 (1.4)
	14.0	121.5	99	115.8	99	-5.7 (-4.7)
	16.6	113.2	99	108.3	99	-4.9 (-4.3)
12000	8.5	142.8	99	145.9	99	3.1 (2.1)
	10.0	138.8	99	139.5	99	0.7 (0.5)
	11.5	132.2	99	133.9	99	1.7 (1.3)
	14.0	122.7	99	116.9	99	-5.8 (-4.7)
	16.6	114.5	99	109.3	99	-5.2 (-4.5)

Table 6.14. Comparison of Maximum Span Lengths for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	Standard		LRFD		Span Difference ft. (%)
		Max. Span (ft.)	**No. Strands**	**Max. Span** (ft.)	**No. Strands**	
6000	8.5	115.1	40	120.9	43	5.9 (5.1)
	10.0	111.2	40	117.1	44	5.9 (5.3)
	11.5	108.1	42	113.6	45	5.5 (5.1)
	14.0	103.5	45	106.3	53	2.8 (2.7)
	16.6	99.4	49	95.8	48	-3.6 (-3.6)
8000	8.5	131.8	55	138.3	59	6.5 (4.9)
	10.0	127.3	55	133.8	61	6.5 (5.1)
	11.5	123.5	57	129.6	62	6.2 (5.0)
	14.0	117.9	62	118.6	72	0.7 (0.6)
	16.6	112.7	66	101.9	55	-10.8 (-9.6)
10000	8.5	145.5	71	152.9	77	7.4 (5.1)
	10.0	140.5	70	147.9	80	7.4 (5.3)
	11.5	136.2	74	143.2	83	7.0 (5.1)
	14.0	130.0	81	127.9	94	-2.1 (-1.6)
	16.6	123.7	88	110.1	68	-13.7 (-11.1)
12000	8.5	157.0	89	162.8	97	5.8 (3.7)
	10.0	151.8	88	155.9	99	4.1 (2.7)
	11.5	146.1	93	149.5	99	3.5 (2.4)
	14.0	136.9	99	129.5	99	-7.4 (-5.4)
	16.6	127.5	99	120.9	99	-6.6 (-5.2)

6.5.2.2 0.5 in. Diameter Strands

For designs using 0.5 in. diameter strands, girder spacing less than or equal to 11.5 ft., and concrete strengths in the range of 6000 to 10000 psi (range where concrete strength works efficiently with the U54 beam under both codes), LRFD designs result in an increase in maximum span up to 6.5 ft. (5.2 percent) (see Table 6.13). This value varies slightly with the concrete strengths and also with the girder spacings. Note that for the maximum effective concrete strength of 10000 psi, the maximum increase in span length is limited to 3.8 ft. (2.7 percent). For girder spacings more than 11.5 ft., the straight comparison indicates two different trends. For a concrete strength of 6000 psi, LRFD designs result in an increase in maximum spans up to 4.0 ft. (4.0 percent). This value varies with the girder spacings and is reduced to 1.0 ft. (1.0 percent) for a girder spacing of 16.6 ft. (see Table 6.13). For a concrete strength of 8000 psi (maximum effective concrete strength), LRFD designs result in a decrease in maximum spans up to 2.6 ft. (2.4 percent). This value varies with the girder spacings and is reduced to 0.9 ft. (0.8 percent) for the girder spacing of 14 ft., as shown in Table 6.13.

Table 6.15. Maximum Difference in Maximum Span Lengths for LRFD Relative to Standard Specifications.

Girder Spacing (ft.)	**Strand Diameter = 0.5 in.**				**Strand Diameter = 0.6 in.**			
	6000 psi	8000 psi	10000 psi	12000 psi	6000 psi	8000 psi	10000 psi	12000 psi
$S \leq 11.5$	5.6 ft. (5.1%)	6.5 ft. (5.2%)	3.8 ft. (2.7%)	---	5.9 ft. (5.3%)	6.5 ft. (5.1%)	7.4 ft. (5.3%)	5.8 ft. (3.7%)
$S > 11.5$	4.0 ft. (4.0%)	-2.6 ft. (-2.4%)	---	---	-3.6 ft. (-3.6%)	-10.8 ft. (-9.6%)	13.7 ft. (-11%)	-7.4 ft. (-5.4%)

6.5.2.3 0.6 in. Diameter Strands

For designs using 0.6 in. diameter strands, girder spacing less than 11.5 ft., and concrete strengths in the range of 6000 to 12000 psi, LRFD designs result in an increase in maximum span up to 5.9 ft. (5.3 percent) (see Table 6.15). This value varies slightly with the concrete strength classes and also with the girder spacings. Note that for the maximum effective concrete strength of 12000 psi, the maximum increase in span length is limited to 5.8 ft. (3.7 percent).

For girder spacings more than 11.5 ft., the straight comparison indicates that for concrete strengths from 6000 to 12000 psi, LRFD designs result in a decrease in maximum span up to 13.7 ft. (11 percent). In general, this value varies with the concrete strength classes and girder spacings. Shorter maximum spans (up to 11 percent) were obtained under the LRFD Specifications for the cases where maximum span lengths are limited by either the maximum number of strand positions in the U54 cross section (99) or by the release stresses at the beam ends.

6.5.3 Comparison of Number of Strands

Tables 6.16 through 6.20 show differences in the number of strands required for span lengths from 90 ft. to the maximum spans designed under the LRFD and the Standard Specifications for 0.6 in. diameter strands. Each table shows the designs for a different girder spacing. The difference in the number of strands for maximum spans is not reported since the number of strands for different spans cannot be compared. For girder spacings less than or equal to 11.5 ft. and for the same span, the LRFD designs required between one and five fewer strands than for the designs using the Standard Specifications. However, for girder spacings greater than 11.5 ft., one to 18 more strands were required for a given span length for the LRFD designs.

The effect of the 0.8 factor included in the LRFD Service III limit state compared with the 1.0 factor considered in the Standard Specifications should result in a reduction of strands required for the same load requirements. However, more strands are needed for girder spacings greater than 11.5 ft. This larger number of strands can be explained by the larger LRFD live load demands. In addition, for U54 Beams designs using the LRFD Specifications, a conservative live load DF found using the lever rule was used for girder spacings greater than 11.5 ft. because the DF given in the specifications was not applicable for larger girder spacings.

Table 6.16. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	**No Strands**	**Length (ft.)**	**No. Strands**	
6000	90	23	90	21	-2
	100	28	100	27	-1
	110	36	110	34	-2
	115.1	40	120	42	-
	-	-	120.9	43	-
8000	90	23	90	20	-3
	100	27	100	25	-2
	110	35	110	32	-3
	120	43	120	40	-3
	130	53	130	50	-3
	131.8	55	138.3	59	-
10000	90	23	90	20	-3
	100	27	100	24	-3
	110	34	110	31	-3
	120	42	120	39	-3
	130	51	130	48	-3
	140	63	140	59	-4
	145.5	71	150	73	-
	-	-	152.9	77	-
12000	90	23	90	20	-3
	100	27	100	24	-3
	110	33	110	30	-3
	120	41	120	38	-3
	130	50	130	47	-3
	140	61	140	58	-3
	150	76	150	71	-5
	157.0	89	160	89	-
	-	-	162.8	97	-

Table 6.17. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 10 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	24	90	23	-1
	100	30	100	30	0
	110	38	110	38	0
	111.2	40	117.1	44	-
8000	90	23	90	22	-1
	100	29	100	28	-1
	110	37	110	36	-1
	120	46	120	45	-1
	127.3	55	130	56	-
	-	-	133.8	61	-
10000	90	23	90	22	-1
	100	28	100	27	-1
	110	36	110	35	-1
	120	45	120	44	-1
	130	55	130	54	-1
	140	69	140	68	-1
	140.5	70	147.9	80	-
12000	90	23	90	22	-1
	100	27	100	26	-1
	110	35	110	34	-1
	120	44	120	43	-1
	130	54	130	53	-1
	140	67	140	66	-1
	150	84	150	82	-2
	151.8	88	155.9	99	-

Table 6.18. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 11.5 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	27	90	25	-2
	100	34	100	33	-1
	108.1	42	110	42	-
	-	-	113.6	45	-
8000	90	26	90	24	-2
	100	33	100	31	-2
	110	42	110	40	-2
	120	53	120	50	-3
	123.5	57	129.6	62	-
10000	90	26	90	24	-2
	100	32	100	30	-2
	110	41	110	39	-2
	120	51	120	49	-2
	130	64	130	61	-3
	136.4	74	140	77	-
12000	90	26	90	24	-2
	100	31	100	29	-2
	110	40	110	38	-2
	120	50	120	48	-2
	130	62	130	60	-2
	140	79	140	75	-4
	146.1	93	149.5	99	-

Table 6.19. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 14 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	32	90	35	3
	100	42	100	46	4
	103.5	45	106.3	53	-
8000	90	31	90	33	2
	100	40	100	44	4
	110	51	110	58	7
	117.9	62	118.6	72	-
10000	90	30	90	32	2
	100	39	100	43	4
	110	49	110	56	7
	120	63	120	74	11
	130	81	127.9	94	-
12000	90	30	90	32	2
	100	37	100	42	5
	110	48	110	54	6
	120	61	120	72	11
	130	79	129.5	99	-
	136.9	99	-	-	-

Table 6.20. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 16.6 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	No. Strands	Length (ft.)	No. Strands	
6000	90	38	90	41	3
	99.4	49	95.8	48	-1
8000	90	36	90	40	4
	100	48	100	50	2
	110	62	101.9	55	-
	112.7	66	-	-	-
10000	90	35	90	38	3
	100	46	100	52	6
	110	60	110	69	9
	120	79	110.1	68	-
	123.7	88	-	-	-
12000	90	34	90	37	3
	100	45	100	50	5
	110	58	110	67	9
	120	77	120	95	18
	127.5	99	120.9	99	-

6.6 STRESSES AT TRANSFER AND TRANSFER LENGTH

Results for the U54 beam designs indicated that the concrete tensile stress at transfer is critical for maximum spans with the widest girder spacings (16.6 ft.) designed under the LRFD Specifications. For the parametric study, the allowable tensile stress at release was taken as the highest limit ($7.5\sqrt{f'_{ci}}$ for Standard designs and $6.96\sqrt{f'_{ci}}$ for LRFD designs, where f'_{ci} is in psi units). This criteria was selected to be consistent with the TxDOT design software, PSTRS14, rather than using the lower limit of the minimum of $3\sqrt{f'_{ci}}$ (where f'_{ci} is in psi units), or 200 psi provided by both AASHTO Specifications when no additional bonded reinforcement is used. Therefore, bonded reinforcement is necessary at the beam ends for the designs in this study. The allowable compressive stress at release was not studied, and the limit is consistent with that given in the Standard and LRFD Specifications.

The parametric study uses the same approach as used in PSTRS14 program, where stresses at the beam ends were determined assuming the strands develop instantaneously after the debonded length. In this case, the strand transfer length is conservatively assumed to be zero. However, the AASHTO Specifications specify that the transfer length is 60 strand diameters.

To assess the impact of this conservative assumption for the transfer length, researches conducted additional analysis for several critical cases was conducted. In addition, they evaluated the impact of the lower tensile stress limit at release was evaluated. The cases considered were the maximum spans using 0.6 in diameter strands for both specifications. The allowable tensile stress at transfer specified as the minimum of $3\sqrt{f'_{ci}}$, or 200 psi, results in the use of 200 psi, which seems more appropriate for normal strength concrete (up to 6000 psi). A limit of $3\sqrt{f'_{ci}}$ was used in this evaluation because the 200 psi limit would dramatically reduce the span lengths for higher strength concrete.

6.6.1 Impact on the Controlling Limit States

6.6.1.1 Standard Specifications

Table 6.21 shows the impact of the allowable release stresses and transfer length on the controlling limit states for maximum span lengths designed using the AASHTO Standard Specifications and 0.6 in. diameter strands. For designs with an allowable tensile stress at transfer of $7.5\sqrt{f'_{ci}}$, no differences were found on the controlling limit states for maximum spans when transfer lengths of 0 and 60 strand diameters were used. Most of the maximum spans were controlled by the compressive concrete stress due to total dead load. For designs with an allowable tensile stress at transfer of $3\sqrt{f'_{ci}}$ and a transfer length of 0 or 60 strand diameters, the tensile stresses at transfer are critical. When the transfer length was considered to be zero, most of the maximum span lengths were controlled by the tensile stress at the beam ends at release. This limited the number of strands, except for the designs using girder spacings of 8.5 and 10 ft. at the lowest concrete strength (6000 psi). Moreover, maximum spans with the widest girder spacing (16.6 ft.) were also controlled by the ultimate moment strength. When the transfer length was considered as 60 strand diameters, only maximum spans with wider girder spacings (14 and 16.6 ft.) were controlled by the tensile stress at the beam ends at release. In this case, the ultimate moment strength did not control the maximum span length.

Table 6.21. Controlling Limit States for Maximum Spans for Different Allowable Release Stresses and Transfer Lengths (AASHTO Standard Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit States			
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$		$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$	
		*L*transfer = 0 (This Study)	*L*transfer = 60 ϕ	*L*transfer = 0	*L*transfer = 60 ϕ
6000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(t) Total Load**
	16.6	f(c) Total Dead Load	f(c) Total Dead Load	Flexural Strength**	f(t) Total Load**
8000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	(f(c) T D L & f(t)_T L)**	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	14.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(t) Total Load**
	16.6	f(c) Total Dead Load	f(c) Total Dead Load	Flexural Strength**	f(t) Total Load**
10000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load & **
	14.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(t) Total Load**
	16.6	f(c) Total Dead Load	f(c) Total Dead Load	Flexural Strength**	f(t) Total Load**
12000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	14.0	f(t) Total Load[99]	f(t) Total Load[99]	f(t) Total Load**	f(t) Total Load**
	16.6	f(t) Total Load[99]	f(t) Total Load[99]	Flexural Strength**	f(t) Total Load**

Notes : See Table 6.2 for Limit State Notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

6.6.1.2 LRFD Specifications

Table 6.22 shows the impact of the allowable release stress and transfer length on the controlling limit states for maximum spans for designs using the LRFD Specifications and 0.6 in. diameter strands. For designs with an allowable tensile stress at transfer of $6.96\sqrt{f'_{ci}}$, a small difference was found in the controlling limit states for maximum span lengths when transfer lengths of zero and 60 strand diameters were used. In the parametric study where zero transfer length was considered, Maximum span lengths with the wider girder spacings (14 and 16.6 ft.) were controlled by the stresses at transfer because this limit state affected the number of strands that could be used. More specifically, the compressive stress controlled maximum spans with girder spacing of 14 ft., and the tensile stress controlled maximum spans with girder spacing of 16.6 ft.

Table 6.22. Controlling Limit States for Maximum Spans for Different Allowable Release Stresses and Transfer Lengths (AASHTO LRFD Specifications, Strand Diameter = 0.6 in.)

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit States			
		$f_t = 6.96\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$		$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$	
		*L*transfer = 0 (This Study)	*L*transfer = 60 ϕ	*L*transfer = 0	*L*transfer = 60 ϕ
6000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	14.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load**	f(t) Total Load**
	16.6	f(t) Total Load**	f(t) _Total Load*	Flexural Strength**	f(t) Total Load**
8000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	14.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load**	f(t) Total Load**
	16.6	f(t) Total Load**	f(t) Total Load*	Flexural Strength**	f(t) Total Load**
10000	8.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load	f(t) Total Load**	f(t) Total Load**
	14.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load**	f(t) Total Load**
	16.6	f(t) Total Load**	f(t) Total Load*	Flexural Strength**	f(t) Total Load**
12000	8.5	f(t) T L/ f (c) T D L	f(t) T L / f (c)_T D L	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(t) Total Load^{99}	f(t) Total Load^{99}	f(t) Total Load**	f(t) Total Load^{99}
	11.5	f(t) Total Load^{99}	f(t) Total Load^{99}	f(t) Total Load**	f(t) Total Load^{99}
	14.0	f(t) Total Load^{99}	f(t) Total Load^{99}	f(t) Total Load**	f(t) Total Load**
	16.6	f(t) Total Load^{99}	f(t) Total Load^{99}	Flexural Strength**	f(t) Total Load**

Note: See Table 6.2 for limit state notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

When a transfer length of 60 strand diameters is considered, the controlling limit states are the same as in the parametric study, except for the maximum span lengths with a 16.6 ft girder spacing. In this case, the controlling limit state at the beam ends is the compressive stress instead of the tensile stress at release.

For designs with the lower tensile stress limit at transfer of $3\sqrt{f'_{ci}}$, the tensile stresses at transfer are critical. For both specifications, when the transfer length is taken as zero, most of the maximum span lengths are controlled by the tensile stress at the beam ends at release, except for designs using girder spacings of 8.5 and 10 ft. at the lowest concrete strength (6000 psi). Maximum spans with the widest girder spacing, 16.6 ft., were also controlled by the ultimate moment strength for all concrete strengths. When the transfer length was considered as 60 strand diameters, only maximum spans with wider girder spacings (14 and 16.6 ft.) were controlled by the tensile stress at the beam ends at release. For this transfer length, the ultimate moment strength does not control any maximum span length.

6.6.2 Impact on Maximum Span Lengths

6.6.2.1 Standard Specifications

Table 6.23 shows the impact of allowable release stress and transfer length on the maximum span lengths for designs using the Standard Specifications and 0.6 in. diameter strands. The percentage differences noted are relative to the parametric study.

Table 6.23. Maximum Span Lengths for Different Allowable Release Stresses and Transfer Lengths (AASHTO Standard Specifications, Strand Diameter = 0.6 in.).

f'_c	Girder Spacing	Maximum Span Lengths						
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$			$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$			
		*L*transfer=0	*L*transfer=60 ϕ		*L*transfer=0		*L*transfer=60 ϕ	
(psi)	(ft.)	Max. Span (This Study) (ft.)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)
6000	8.5	115.1	115.1	-	115.1	-	115.1	-
	10.0	111.2	111.2	-	111.2	-	111.2	-
	11.5	108.1	108.1	-	102.8	-5.3 (-4.9)	108.1	-
	14.0	103.5	103.5	-	87.4	-16.1 (-15.6)	97.3	-6.2 (-5.9)
	16.6	99.4	99.4	-	72.5	-27.1 (-27.1)	84.7	-14.7 (-14.9)
8000	8.5	131.8	131.8	-	131.8	-	131.8	-
	10.0	127.3	127.3	-	127.1	-0.2 (-0.2)	127.3	-
	11.5	123.5	123.5	-	115.9	-7.6 (-6.1)	123.5	-
	14.0	117.9	117.9	-	93.6	-24.3 (-20.6)	104.7	-13.2 (-11.2)
	16.6	112.7	112.7	-	79.9	-32.8 (-29.1)	90.2	-22.5 (-20.0)
10000	8.5	145.5	145.5	-	142.3	-3.2 (-2.2)	145.5	-
	10.0	140.5	140.4	-	134.6	-5.9 (-4.2)	140.5	-
	11.5	136.2	136.2	-	121.8	-14.5 (-10.6)	136.3	-
	14.0	130.0	130.0	-	96.9	-33.1 (-25.4)	113.0	-17.0 (-13.1)
	16.6	123.7	123.7	-	85.2	-38.6 (-31.2)	95.4	-28.3 (-22.9)
12000	8.5	157.0	157.0	-	147.2	-9.7 (-6.2)	157.0	-
	10.0	151.8	151.8	-	138.8	-13.0 (-8.6)	151.8	-
	11.5	146.1	146.0	-	126.3	-19.7 (-13.5)	146.0	-
	14.0	136.9	136.8	0.1 (-0.1)	103.3	-33.6 (-24.5)	121.2	-15.7 (-11.4)
	16.6	127.5	127.5	-	90.2	-37.3 (-29.3)	99.1	-28.4 (-22.3)

Notes: See Table 6.2 for limit state notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameter from debonded length toward midspan)

For designs using the Standard Specifications and the allowable tensile stress at release of $7.5\sqrt{f'_{ci}}$, the use of a transfer length of 60 strand diameters does not have an impact on the maximum span lengths. However, for designs using the Standard Specifications where the allowable tensile stress at release was $3\sqrt{f'_{ci}}$, increases in maximum span lengths up to 19.7 ft. (15.6 percent) were found when the transfer length of 60 strand diameters was used versus a transfer length of zero.

Table 6.23 shows decreases in maximum span lengths up to 19.7 ft. (13.5 percent) for girder spacings less than or equal to 11.5 ft. and up to 38.6 ft. (31.2 percent) for girder spacings greater than 11.5 ft. when the limit for the tensile stress changes from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with zero transfer length). The same table shows decreases in maximum span lengths up to 28.3 ft. (22.9 percent) for girder spacings greater than 11.5 ft. when the limit for the tensile stress changes from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with transfer length of 60 strand diameters). No differences were found for girder spacings less than 11.5 ft.

6.6.2.2 LRFD Specifications

Table 6.24 shows the impact of allowable release stress and transfer length on the maximum span lengths for designs using the AASHTO LRFD Specifications and 0.6 in. diameter strands. Some different trends were observed for the LRFD designs versus the Standard designs. For designs using the LRFD Specifications with the allowable tensile stress at transfer of $6.96\sqrt{f'_{ci}}$, the use of a transfer length of 60 strand diameters resulted in increases up to 9.3 ft. (9.1 percent) in the maximum span lengths only for the widest girder spacing (16.6 ft.). For designs using the LRFD Specifications where the allowable tensile stress at transfer was $3\sqrt{f'_{ci}}$, percentage increases in maximum span lengths up to 16.6 ft. (23.6 percent) were found when the transfer length was changed from 0 to 60 strand diameters.

Table 6.24 shows decreases up to 19.9 ft. (13.3 percent) in the maximum span length for girder spacings less than or equal to 11.5 ft. and up to 36.8 ft. (30.4 percent) for girder spacings greater than 11.5 ft. when the limit for the tensile stress changes from $6.96\sqrt{f'_{ci}}$ to $3\sqrt{f'_{ci}}$ with zero transfer length. The same table shows decreases in maximum span lengths up to 28 ft. (23.2 percent) for girder spacings greater than 11.5 ft. when the limit for the tensile stress goes from

$6.96\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with transfer length of 60 strand diameters). No significant differences were found for girder spacings less than 11.5 ft.

Table 6.24. Maximum Span Lengths for Different Allowable Release Stresses and Transfer Lengths (AASHTO LRFD Specifications - Strand Diameter = 0.6 in.).

f'_c	Girder Spacing	Maximum Span Lengths						
		$f_t = 6.96\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$			$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$			
		*L*transfer=0	*L*transfer=60 ϕ		*L*transfer=0		*L*transfer=60 ϕ	
(psi)	(ft.)	Max. Span (This Study) (ft.)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)
6000	8.5	120.9	120.9	-	120.9	-	120.9	-
	10.0	117.1	117.0	-	117.1	-	117.0	-
	11.5	113.6	113.5	-	111.2	-2.4 (-2.1)	113.7	0.1 (0.2)
	14.0	106.3	107.0	0.7 (0.6)	82.5	-23.7 (-22.3)	92.2	-14.1 (-13.3)
	16.6	95.8	99.3	3.5 (3.6)	65.9	-29.9 (-31.3)	81.4	-14.4 (-15.0)
8000	8.5	138.3	138.3	-	138.3	-	138.3	-
	10.0	133.8	133.8	-	130.8	3.0 (-2.2)	133.9	0.1 (0.1)
	11.5	129.6	129.7	0.1 (0.1)	121.4	-8.2 (-6.3)	129.7	0.1 (0.1)
	14.0	118.6	119.6	1.0 (0.8)	88.4	-30.2 (-25.5)	96.3	-22.3 (-18.8)
	16.6	101.9	111.2	9.3 (9.1)	74.7	-27.2 (-26.7)	85.5	-16.4 (-16.1)
10000	8.5	152.9	152.9	-	149.0	-3.9 (-2.6)	152.9	-
	10.0	147.9	147.9	-	136.1	-11.8 (-8.0)	147.9	-
	11.5	143.2	143.2	-	125.4	-17.8 (-12.5)	140.6	-2.6 (-1.8)
	14.0	127.9	128.5	0.6 (0.5)	92.7	-35.2 (-27.5)	101.0	-26.9 (-21.0)
	16.6	110.1	119.6	9.5 (8.6)	79.5	-30.5 (-27.7)	89.4	-20.7 (-18.8)
12000	8.5	162.8	162.7	-	162.1	-0.7 (-0.4)	162.0	-0.8 (-0.5)
	10.0	155.9	155.8	-	141.0	-14.9 (-9.6)	155.8	-
	11.5	149.5	149.5	-	129.6	-19.9 (-13.3)	149.5	-
	14.0	129.5	129.5	-	95.6	-34 (-26.2)	106.1	-23.4 (-18.1)
	16.6	120.9	120.9	-	84.1	-36.8 (-30.4)	92.9	-28.0 (-23.2)

Notes: See Table 6.2 for limit state notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

6.7 EFFECT OF ALLOWABLE TENSILE STRESS AT SERVICE

Prior to completion of Phase 3 of this project (Hueste et al. 2003c), a preliminary assessment of the impact of revising critical design criteria with the objective of increasing the economy of HSC prestressed girders was addressed as part of this study. As noted earlier, current specifications provide allowable stresses that were developed based on the mechanical properties of NSC of 6000 psi or less. These values that are traditionally conservative for standard designs using NSC may not be appropriate for HSC designs. Because prestressed concrete design is often governed by the allowable stresses, researchers studied the effects of the allowable stresses on the required number of strands and consequently on the span capability.

The results of the parametric study showed that the allowable tensile stress limit is critical because it controls the designs (number of strands required) in most cases for longer spans. Based on review of current allowable stresses (see Section 2.5) and considering the HSC Louetta Bridge design (Ralls 1995), an allowable tensile stress of $7.5\sqrt{f'_c}$ (where f'_c is in psi units) was selected for this preliminary assessment. This stress limit can also be compared to the modulus of rupture for HSC determined in Phase 1 of this study (Hueste et al. 2003b), which was found to have a best-fit equation of $10\sqrt{f'_c}$, with a lower bound value of about $8\sqrt{f'_c}$. Note that Phase 3 focused on assessing the impact on field curing conditions on the compressive strength and modulus of rupture of HSC (Hueste et al. 2003c). However, these results were not available at the time this phase of the project took place.

The parametric study showed that HSC prestressed bridge girder designs are often controlled by the compressive stress limits. In addition, Phase 1 of this study showed that for HSC produced by Texas precasters, the actual concrete compressive strength at service is typically greater than specified, where the ratio of the actual to specified f'_c ranged from 1.01 to 1.89 (Hueste et al. 2003b). However, increases in the compressive stress limits were not selected for evaluation in this study, and the allowable compressive stress was maintained as $0.45\,f'_c$ as specified in the LRFD Specifications and in ACI 318-02. The reason for this is that the current limits for the compressive stresses were established to limit excessive creep, camber, or other local strains. The compressive stress limits for sustained loads ($0.4\,f'_c$ to $0.45\,f'_c$) are generally in the linear range of behavior for NSC. An increase in the stress limit to $0.6\,f'_c$ is allowed for load cases including transient loads. These limits were developed for NSC, and more studies are needed to evaluate whether these limits are applicable to HSC. Assuming that the same coefficients are appropriate for the compressive stress limits for HSC prestressed members, it is not conservative to assume an overstrength will be provided in the design phase, because production practices may change among precasters over time and this overstrength is not a requirement. Potentially, the actual strength gain can be utilized by tailoring designs based on strength data for a typical concrete mixture used by the selected precaster. However, the precaster may not be identified in the initial design stage and so this may not always be practical.

6.7.1 Impact on the Controlling Limit States

Tables 6.25 through 6.29 provide controlling limit states for different allowable tensile stresses at service for U54 Beams with spans from 90 ft. to maximum span lengths at 10 ft. intervals. A separate table is provided for each girder spacing considered. Different concrete classes are considered, and all cases are for 0.6 in. diameter strands with designs according to the LRFD Specifications. The controlling limit states are the limit state that dictates the required number of strands or limits the maximum span.

Results showed that for shorter spans (in several cases up to 100 ft., and 110 ft. in one case), allowing a higher tensile stress has an impact on the ultimate strength of the beams because increasing the tensile stress limit resulted in a reduction of the number of strands required for a given span length. As girder spacings decrease and concrete strengths increase, the flexural strength becomes more critical. For a given longer span length, except for maximum span lengths, the use of $f_t = 7.5\sqrt{f'_c}$ resulted in designs that were controlled by the tensile stress limit, as was the case when using $f_t = 6\sqrt{f'_c}$. However, fewer strands were required when using the higher tensile stress limit (see Sections 6.7.2 and 6.7.3).

Table 6.25. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 8.5 ft.).

f'_c	Controlling Limit State			
(psi)	**Length** (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	120.9	f(c) Total Dead Load	120.3	f(c) Total Dead Load
8000	90	Flexural Strength / f(t)_T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	138.3	f(c) Total Dead Load	137.9	f(c) Total Dead Load
10000	90	Flexural Strength	90	Flexural Strength
	100	Flexural Strength / f(t)_T L	100	Flexural Strength
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	152.9	f(c) Total Dead Load	152.6	f(c) Total Dead Load
12000	90	Flexural Strength	90	Flexural Strength
	100	Flexural Strength / f(t)_T L	100	Flexural Strength
	110	f(t) Total Load	110	Flexural Strength
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	160	f(t) Total Load	160	f(t) Total Load
	162.8	f(t) T L / f(c) T D L	164.5	f(t) T L / f(c) T D L

Table 6.26. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 10 ft.).

f'c (psi)	**Controlling Limit State**			
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	117.1	f(c) Total Dead Load	116.4	f(c) Total Dead Load
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	133.8	f(c) Total Dead Load	133.5	f(c) Total Dead Load
10000	90	Flexural Strength	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	147.9	f(c) Total Dead Load	147.6	f(c) Total Dead Load
12000	90	Flexural Strength	90	Flexural Strength
	100	Flexural Strength / f(t) T L	100	Flexural Strength
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	155.9	f(t) Total Load[99]	158.0	f(t) T L / f(c) T D L

Table 6.27. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 11.5 ft.).

f'_c (psi)	Controlling Limit State			
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	113.6	f(c) Total Dead Load	112.9	f(c) Total Dead Load
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	129.6	f(c) Total Dead Load	129.4	f(c) Total Dead Load
10000	90	Flexural Strength	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	143.2	f(c) Total Dead Load	143.0	f(c) Total Dead Load
12000	90	Flexural Strength	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	149.5	f(t) Total Load[99]	150	f(t) Total Load
	-	-	152.2	f(t) T L / f(c) T D L

Table 6.28. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 14 ft.).

f'_c (psi)	Controlling Limit States			
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	Length (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	106.3	f(t) Total Load*	108.3	f(t) Total Load*
8000	90	f(t) Total Load	90	Flexural Strength
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	118.6	f(t) Total Load*	120	f(t) Total Load
	-	-	121.4	f(t) Total Load*
10000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	127.9	f(t) Total Load*	130	f(t) Total Load
	-	-	130.3	f(t) Total Load*
12000	90	Flexural Strength	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	129.5	f(t) Total Load[99]	130	f(t) Total Load
	-	-	132.2	f(t) Total Load[99]

Table 6.29. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 16.6 ft.).

f'_c (psi)	Controlling Limit States			
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	Length (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90.0	f(t) Total Load	90.0	f(t) Total Load
	95.8	f(t) Total Load**	98.5	f(t) Total Load**
8000	90.0	f(t) Total Load	90.0	Flexural Strength / f(t) T L
	100.0	f(t) Total Load	100.0	f(t) Total Load
	101.9	f(t) Total Load**	107.5	f(t) Total Load**
10000	90.0	f(t) Total Load	90.0	Flexural Strength
	100.0	f(t) Total Load	100.0	f(t) Total Load
	110.0	f(t) Total Load	110.0	f(t) Total Load
	110.1	f(t) Total Load**	115.1	f(t) Total Load**
12000	90.0	f(t) Total Load	90.0	Flexural Strength
	100.0	f(t) Total Load	100.0	f(t) Total Load
	110.0	f(t) Total Load	110.0	f(t) Total Load
	120.0	f(t) Total Load	120.0	f(t) Total Load
	120.9	f(t) Total Load[99]	123.4	f(t) Total Load[99]

6.7.2 Impact on the Number of Strands

Tables 6.30 through 6.34 show the differences for the number of strands required for spans from 90 ft. to maximum span lengths at 10 ft. intervals designed with two different allowable tensile stress limits ($f_t = 6\sqrt{f'_c}$ and $f_t = 7.5\sqrt{f'_c}$) for different concrete classes and girder spacings. The calculations were performed for U54 Beams with 0.6 in. diameter strands designed using the LRFD Specifications.

The number of strands required for maximum span lengths is not compared in the tables above but is discussed in Section 6.7.4. For the same span and for girder spacings up to 11.5 ft., designs using $f_t = 7.5\sqrt{f'_c}$ required between one to seven fewer strands than for designs using $f_t = 6\sqrt{f'_c}$. For the same span and for girder spacings greater than 11.5 ft., designs using $f_t = 7.5\sqrt{f'_c}$ required between 1 and 11 fewer strands than for designs using $f_t = 6\sqrt{f'_c}$. The percentage reduction in the number of strands when using the larger allowable tensile stress ranged from 0 to 12 percent.

Table 6.30. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 8.5 ft.).

f'$_c$ (psi)	**Number of Strands**				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		**Difference No. Strands**
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	21	90	20	-1
	100	27	100	25	-2
	110	34	110	31	-3
	120	42	120	39	-3
	120.9	43	120.3	40	-
8000	90	20	90	20	-
	100	25	100	24	-1
	110	32	110	30	-2
	120	40	120	38	-2
	130	50	130	47	-3
	138.3	59	137.9	55	-
10000	90	20	90	20	0
	100	24	100	24	0
	110	31	110	29	-2
	120	39	120	36	-3
	130	48	130	45	-3
	140	59	140	55	-4
	150	73	150	68	-5
	152.9	77	152.6	72	-
12000	90	20	90	20	0
	100	24	100	24	0
	110	30	110	28	-2
	120	38	120	35	-3
	130	47	130	44	-3
	140	58	140	53	-5
	150	71	150	66	-5
	160	89	160	82	-7
	162.8	97	164.5	91	-

Table 6.31. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 10 ft.).

f'_c (psi)	Number of Strands				
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$	**Difference**
6000	90	23	90	22	-1
	100	30	100	27	-3
	110	38	110	35	-3
	117.1	44	116.4	41	-
8000	90	22	90	22	0
	100	28	100	26	-2
	110	36	110	34	-2
	120	45	120	42	-3
	130	56	130	53	-3
	133.8	61	133.5	57	-
10000	90	22	90	22	0
	100	27	100	26	-1
	110	35	110	32	-3
	120	44	120	41	-3
	130	54	130	51	-3
	140	68	140	63	-5
	147.9	80	147.6	75	-
12000	90	22	90	22	0
	100	26	100	26	0
	110	34	110	31	-3
	120	43	120	39	-4
	130	53	130	49	-4
	140	66	140	61	-5
	150	82	150	76	-6

Table 6.32. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 11.5 ft.).

f'_c (psi)	Number of Strands				
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$	**Difference**
6000	90	25	90	23	-2
	100	33	100	31	-2
	110	42	110	39	-3
	113.6	45	112.9	42	-
8000	90	24	90	24	0
	100	31	100	29	-2
	110	40	110	37	-3
	120	50	120	47	-3
	129.6	62	129.4	59	-
10000	90	24	90	24	0
	100	30	100	28	-2
	110	39	110	36	-3
	120	49	120	46	-3
	130	61	130	57	-4
	140	77	140	72	-5
	143.2	83	143.0	77	-
12000	90	24	90	24	0
	100	29	100	28	-1
	110	38	110	35	-3
	120	48	120	44	-4
	130	60	130	55	-5
	140	75	140	70	-5
	149.5	99	150	89	-
	-	-	152.2	97	-

Table 6.33. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 14 ft.).

f'_c (psi)	Number of Strands				
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$	**Difference**
6000	90	35	90	32	-3
	100	46	100	43	-3
	106.3	53	108.3	53	-
8000	90	33	90	32	-1
	100	44	100	41	-3
	110	58	110	54	-4
	118.6	72	120	71	-
	-	-	121.4	73	-
10000	90	32	90	33	1
	100	43	100	40	-3
	110	56	110	52	-4
	120	74	120	69	-5
	127.9	94	130	93	-
	-	-	130.3	94	-
12000	90	32	90	32	0
	100	42	100	38	-4
	110	54	110	50	-4
	120	72	120	66	-6
	129.5	99	130	90	-
	-	-	132.2	99	-

Table 6.34. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 16.6 ft.).

f'_c (psi)	Number of Strands				
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$	**Difference**
6000	90	41	90	39	-2
	95.8	48	98.5	49	-
8000	90	40	90	37	-3
	100	53	100	50	-3
	101.9	55	107	61	-
10000	90	38	90	37	-1
	100	52	100	48	-4
	110	68	110	64	-4
	110.1	68	115.1	74	-
12000	90	37	90	37	0
	100	50	100	46	-4
	110	67	110	62	-5
	120	95	120	84	-11
	120.9	99	123.4	99	-

6.7.3 Impact on the Controlling Limit States for Maximum Span Lengths

Table 6.35 shows the controlling limit states for two different allowable tensile stresses at service (f_t = $6\sqrt{f'_c}$ and f_t = $7.5\sqrt{f'_c}$) for maximum span lengths. The calculations were performed for U54 Beams with 0.6 in. diameter strands designed using the LRFD Specifications. To further study the impact of increasing the allowable tensile strength at service, additional designs were performed using $f_t = 8\sqrt{f'_c}$. Tables C.21 and C.22 in Appendix C show maximum span lengths with their respective number of strands, initial concrete strengths, and controlling limit states designed using $f_t = 8\sqrt{f'_c}$.

Several major trends were observed. First, for concrete strengths up to 10000 psi and for girder spacings less than or equal to 11.5 ft., maximum span lengths were controlled by the same controlling limit states (compressive limit at the intermediate stage) when the limit for the tensile stress at service changes from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$. A reduction of number of strands is possible because the tensile limit was increased. However, maximum span lengths do not increase because maximum span lengths are controlled by the compressive limit at the intermediate state (see Section 6.7.4). The same trend was observed when the tensile stress was increased to f_t = $8\sqrt{f'_c}$ because the tensile stress does not control maximum span lengths for these cases (see Tables C.21 and C.22 of Appendix C).

Table 6.35. Controlling Limit States for Maximum Span Lengths for Different Allowable Tensile Stresses at Service.

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit State	
		$f_t = 6\sqrt{f'_c}$	$f_t = 7.5\sqrt{f'_c}$
6000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(t) Total Load*	f(t) Total Load*
	16.6	f(t) Total Load**	f(t) Total Load**
8000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(t) Total Load*	f(t) Total Load*
	16.6	f(t) Total Load**	f(t) Total Load**
10000	8.5	f(c) Total Dead Load	f(c) Total Dead Load
	10.0	f(c) Total Dead Load	f(c) Total Dead Load
	11.5	f(c) Total Dead Load	f(c) Total Dead Load
	14.0	f(t) Total Load*	f(t) Total Load*
	16.6	f(t) Total Load**	f(t) Total Load**
12000	8.5	f(t) Total Load / f(c) T D L	f(t) Total Load / f(c) T D L
	10.0	f(t) Total Load [99]	f(t) Total Load / f(c) T D L
	11.5	f(t) Total Load [99]	f(t) Total Load / f(c) T D L
	14.0	f(t) Total Load [99]	f(t) Total Load [99]
	16.6	f(t) Total Load [99]	f(t) Total Load [99]

Notes: See Table 6.2 for Limit State Notation
* f(c) at ends during transfer controls, followed by the limit state given
** f(t) at ends during transfer controls, followed by the limit state given

For f'_c up to 10000 psi and for wider girder spacing (14 and 16.6 ft.) where maximum span lengths are previously controlled by either the release compressive limit or the tensile limit, followed by the tensile limit at service, an increase in the tensile limit at service (from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$) resulted in the same controlling limit states but with increases in maximum span lengths. For these cases, an increase in the number of strands was required (see Section 6.7.4). The same trends (controlling limit states) were observed when the tensile stress was increased to $8\sqrt{f'_c}$. Maximum span lengths continued to increase and more strands were required (see Tables C.21 and C.22).

For an f'_c of 12000 psi, two different trends were observed for the controlling limit states when raising the tensile limit at service (from $f_t = 6\sqrt{f'_c}$ to $f_t = 7.5\sqrt{f'_c}$). First, the controlling limit state changed from the tensile limit at service to the compressive limit at the intermediate stage. Second, the tensile limit state at service that controlled for cases when no additional strands can be fit in the U54 beam section remained the same. For all these cases, increases in maximum span lengths were observed. For the case where the tensile stress was increased to $8\sqrt{f'_c}$, the controlling limit state was also the compressive limit at the intermediate stage and the maximum span lengths increased. However, in a number of cases the increase in the tensile stress (beyond $8\sqrt{f'_c}$) not only gave a reduction in the number of strands but also in the maximum span length (see designs for f'_c values up to 10000 psi with girder spacings less than 11.5 ft). Again, the tensile limit state at service controlled for cases when no additional strands can be fit in the U54 beam section (see Tables C.21 and C.22).

6.7.4 Impact on Maximum Span Lengths

Table 6.36 shows maximum span lengths for two different allowable tensile stress limits ($f_t = 6\sqrt{f'_c}$ and $f_t = 7.5\sqrt{f'_c}$). The calculations were performed for U54 Beams with 0.6 in. diameter strands designed using the LRFD Specifications. To further study designs that were controlled by the tensile limit at service, additional designs were performed using $f_t = 8\sqrt{f'_c}$. Maximum span lengths with their respective number of strands, initial concrete strengths, and controlling limit states designed using $f_t = 8\sqrt{f'_c}$ are shown in Tables C.21 and C.22 in Appendix C.

Several major trends were observed. First, for concrete strengths up to 10000 psi and for girder spacings less than or equal to 11.5 ft, where maximum span lengths are controlled by the same controlling limit states (compressive limit at the intermediate stage), the increase in the allowable tensile stress at service from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ actually results in small (up to 0.7 ft. [0.6 percent]) decreases in maximum span lengths. The same trend was observed for designs using $f_t = 8\sqrt{f'_c}$, where decreases in maximum span lengths go up to 0.9 ft. (0.7 percent) (see Tables C.21 and C.22 of Appendix C). These decreases in the maximum span lengths occur because a reduction in the number of required strands results in a small increase in the net compressive stress.

For concrete strengths up to 10000 psi and for wider girder spacings (14 and 16.6 ft.) where maximum span lengths are previously controlled by either the release compressive limit or the release tensile limit, followed by the tensile limit at service, an increase in the tensile limit at service (from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$) resulted in up to a 5.6 ft. (5.5 percent) increase in maximum span lengths. However, more strands were required. These increases in spans can be explained because more strands can be used when the tensile limit at service is increased. The same trend was observed when the tensile limit was raised to $8\sqrt{f'_c}$. In this case increases up to 6.8 ft. (6.7 percent) were determined when providing additional strands.

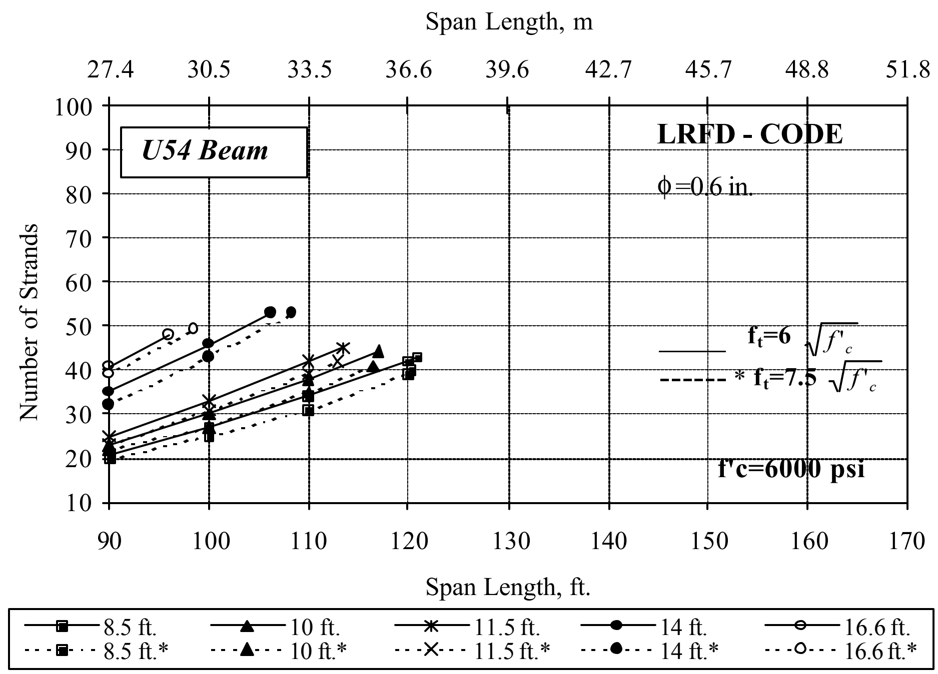
Third, for a concrete strength of 12000 psi, two basic trends were observed on the controlling limit states when raising the tensile limit at service. For girder spacings less than 11.5 ft. where the controlling tensile limit changed from the tensile limit at service to the compressive limit at the intermediate stage (having a small available tensile stress), increases up to 2.7 ft. (1.8 percent) were observed. For wider girder spacings (14 and 16.6 ft.) the controlling tensile limit state (where no additional strands can be fit in the U54 beam section) remained the same and increases up to 2.5 ft. (2.1 percent) were observed. When the tensile stress limit was increased to $8\sqrt{f'_c}$, the first trend explained above was different. For f'_c values up to 10000 psi and girder spacings less than 11.5 ft., the controlling limit state changed from the tensile limit to the compressive limit at the intermediate stage and maximum spans increased up to 3.3 ft. (2.2 percent). When the compressive limit controls designs, increases in the tensile limit beyond $8\sqrt{f'_c}$ would result not only in a reduction of the number of strands but also in a reduction of maximum span lengths. For wider girder spacings, increases up to 3.5 ft. (2.7 percent) were found (see Tables C.21 and C.22).

Table 6.36. Maximum Span Lengths for Different Allowable Tensile Stresses at Service.

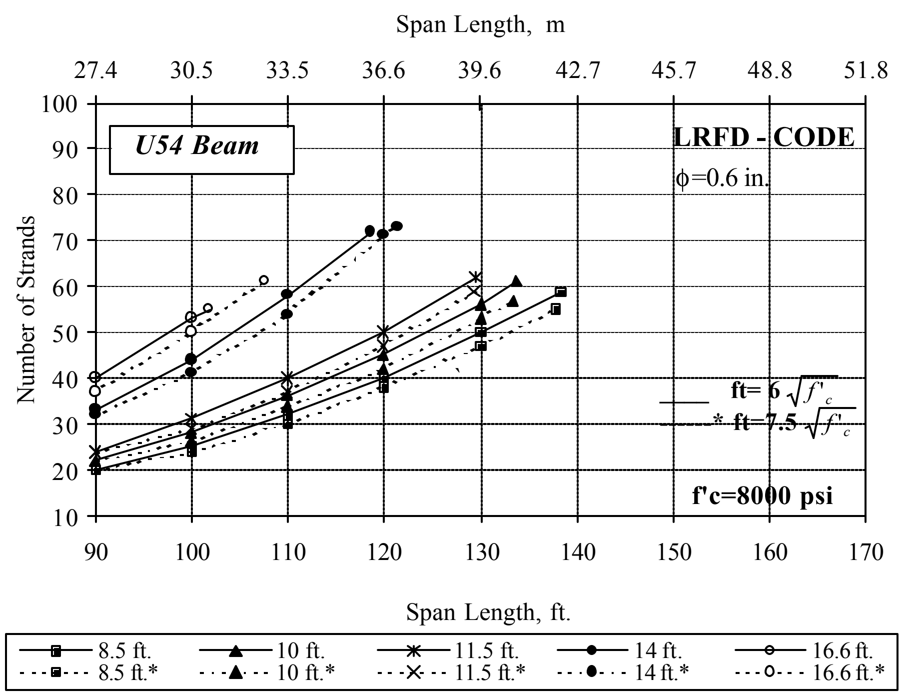
f'_c (psi)	Girder Spacing (ft.)	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Difference Max. Span
		Max. Span (ft.)	No. Strands	Max. Span (ft.)	No. Strands	ft. (%)
6000	8.5	120.9	43	120.3	40	-0.7 (-0.6)
	10.0	117.1	44	116.4	41	-0.7 (-0.6)
	11.5	113.6	45	112.9	42	-0.6 (-0.6)
	14.0	106.3	53	108.3	53	2.1 (1.9)
	16.6	95.8	48	98.5	49	2.7 (2.8)
8000	8.5	138.3	59	137.9	55	-0.4 (-0.3)
	10.0	133.8	61	133.5	57	-0.4 (-0.3)
	11.5	129.6	62	129.4	59	-0.3 (-0.2)
	14.0	118.6	72	121.4	73	2.8 (2.3)
	16.6	101.9	55	107.5	61	5.6 (5.5)
10000	8.5	152.9	77	152.6	72	-0.3 (-0.2)
	10.0	147.9	80	147.6	75	-0.3 (-0.2)
	11.5	143.2	83	143.0	77	-0.2 (-0.2)
	14.0	127.9	94	130.3	94	2.5 (1.9)
	16.6	110.1	68	115.1	74	5.1 (4.6)
12000	8.5	162.8	97	164.5	91	1.7 (1.0)
	10.0	155.9	99	158.0	94	2.1 (1.3)
	11.5	149.5	99	152.2	97	2.7 (1.8)
	14.0	129.5	99	132.2	99	2.6 (2.0)
	16.6	120.9	99	123.4	99	2.5 (2.1)

6.7.5 Span Capability

Figure 6.5 shows the impact on span capability of the U54 beam designed using the AASHTO LRFD Specifications for two different allowable tensile stresses, $6\sqrt{f'_c}$ and $7.5\sqrt{f'_c}$. These figures show the trends for number of strands versus span lengths for different girder spacings and concrete strengths. There are two ways to interpret these results. On the vertical axis, each interval of allowable tensile stress represents a savings of between approximately one to seven strands (11 for one case) for the same span and girder spacing. Researchers found that a reduction in the required f'_c at release (see Tables C.21 and C.22, provided in Appendix C). On the horizontal axis, each interval of allowable tensile stress represents an increase in span capability of approximately 2.5 to 5 ft. for the same number of strands and girder spacing.

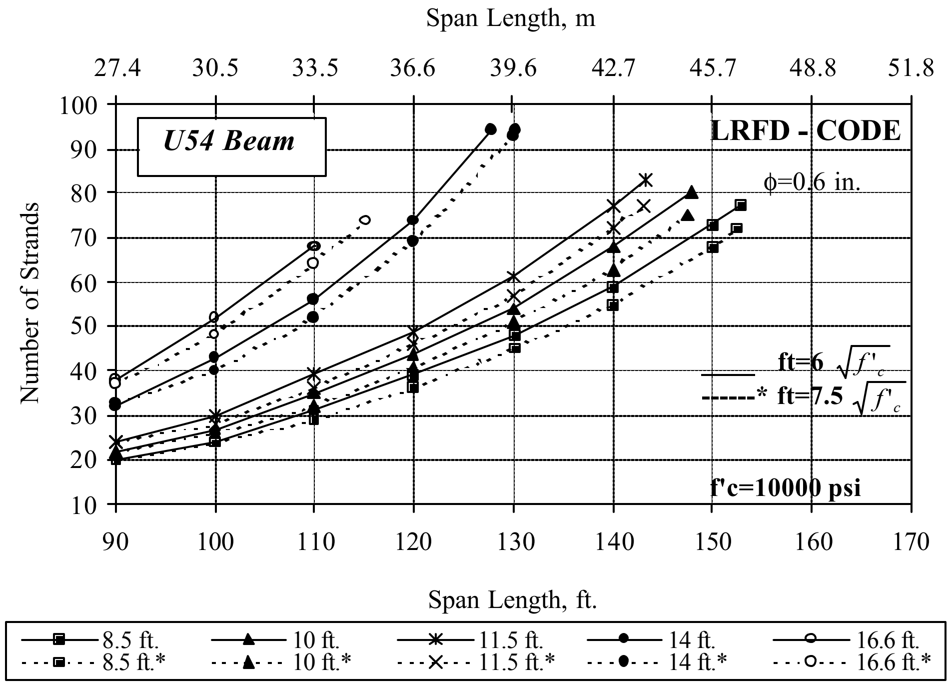


(a) f'_c **= 6000 psi**

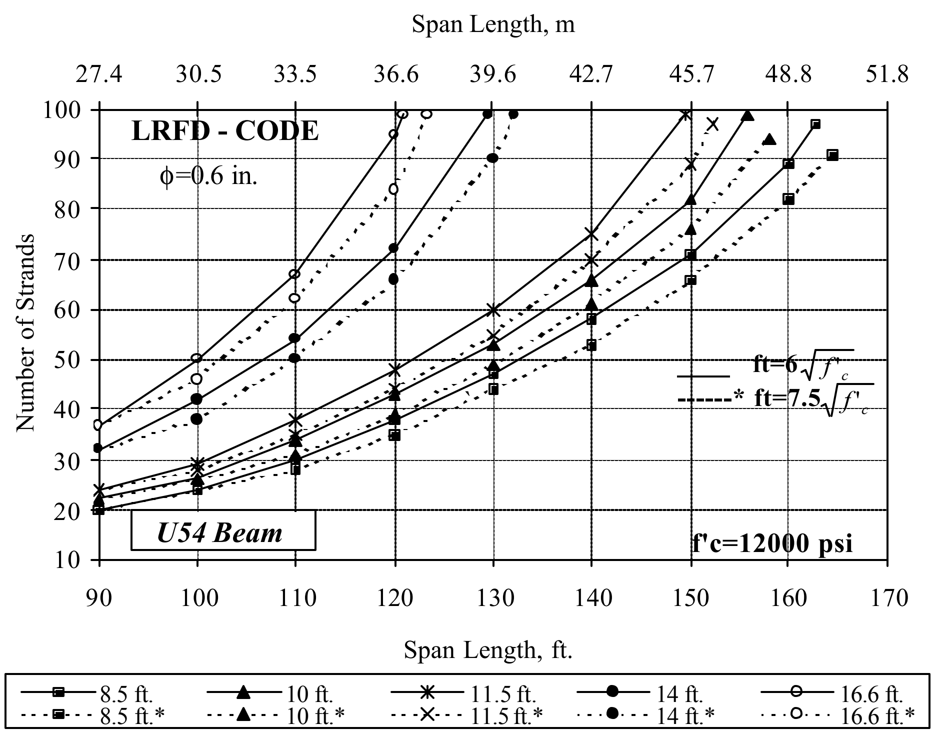


(b) f'_c **= 8000 psi**

Figure 6.5. Number of Strands versus Span Lengths for Different Allowable Tensile Stresses (LRFD Specifications, Strand Diameter = 0.6 in.).



(c) f'_c = 10000 psi



(d) f'_c = 12000 psi

Figure 6.5. Continued.

7 RESULTS FOR TYPE IV BEAMS

7.1 INTRODUCTION

Researchers conducted a parametric study composed of a number of designs using AASHTO Type IV prestressed concrete bridge girders. The main objective was to investigate the controlling limit states and the impact of varying the concrete compression strength of the precast section, strand diameters, girder spacing, and code requirements. Only the flexural limit states (service and ultimate) were included in this study. A summary of the design parameters is given in Table 7.1, and additional details are provided in Section 5.5.

Table 7.1 Summary of Design Parameters.

Parameter	Description and Selected Values
Codes	AASHTO Standard and LRFD Specifications (2002 a,b)
Concrete Strength (psi)	6000, 8000, 10000, and 12000 (f'_{ci} was initially set at 0.75 f'_c, but allowed to vary up to f'_c)
Girder Spacing (ft.)	4.25, 5, 5.75, 7, 8.5, and 9
Spans	90 ft. to maximum span at 10 ft. intervals

For the parametric study, the span lengths were varied from 90 ft. to the maximum span possible for a given set of parameters, using 10 ft. increments. For each of these spans, the most economical design (fewest number of strands) was determined and the corresponding controlling limit state was identified. In the discussion of results, the span lengths are labeled as "shorter spans," "longer spans," and "maximum spans." A shorter span is generally in the range of 90 to 100 ft. long. A longer span is a span that is greater than 100 ft. up to, but not including, the maximum span length. The maximum span length is the length beyond which a flexural limit state would be exceeded, such that for the particular set of parameters the span has been maximized. For every case studied, key design information is available in tables provided in Appendix D. Based on these results, the following sections summarize the findings, with a primary focus on the maximum spans.

7.2 DESCRIPTION OF CONTROLLING LIMIT STATES

A controlling limit state is defined for this study as the flexural design limit state that dictates the required number of strands for a given geometry and demand. In the case of establishing the maximum span length, the controlling limit state is defined as the limit state that would be exceeded if the span was increased. Limit states include satisfying the allowable stresses and required ultimate flexural strength, both at the maximum moment section along the span and at the beam ends. The required number of strands is determined to ensure that the allowable stresses are not exceeded as the beam is loaded from the initial to the final service stage. In addition, the ultimate flexural strength is checked. The required number of strands is computed using a systematic approach that is based on attaining actual stresses as near as possible to the corresponding allowable stresses for the considered load stages to achieve the most economical design (see Section 5.4).

The number of strands and, consequently, span lengths, are primarily controlled by one of the four allowable stresses: compressive and tensile stresses at the beam ends upon release of the prestressing strands, compressive sustained load stresses, and tensile service load stresses or by the required flexural strength at ultimate conditions. The compressive service load stress and the stresses at midspan at release were also considered in the designs, but were not critical. Researchers also considered combinations of the controlling limit states for the cases where temporary allowable stresses at the beam ends or eccentricity limitations initially control the number of strands that may be used, followed by exceeding the allowable stresses for the sustained or service load conditions. According to the limits above, Table 7.2 identifies flexural limit states that control the required number of strands for maximum span lengths for the Type IV girders.

Table 7.2. Controlling Limit States for Type IV Girders.

Controlling Limit State	Description
Flexural Strength	Required flexural strength at ultimate.
f(t) Total Load	The number of strands is controlled by the concrete tensile stress at midspan at the final stage due to total loads (including live loads).
f(c) Total Dead Load	The number of strands is controlled by the concrete compressive stress at midspan at the intermediate stage due to total dead loads (not including live loads).
f(t) Total Load [e]	The number of strands is controlled by the concrete tensile stress at midspan at the final stage due to total loads. Unlike the same limit state defined above, this occurs when an effective eccentricity is used. Additional strands beyond this number do not provide a significant gain in length.
f(t) Total Load*	The number of strands is initially limited by the concrete compressive stress at the beam ends at release, followed by the concrete tensile stress at midspan at the final stage.
f(c) Total Dead Load*	The number of strands is initially limited by the concrete compressive stress at the beam ends at release, followed by the concrete compressive stress at midspan at the intermediate stage due to sustained loads.
f(t) T L / f(c) T D L	The number of strands is initially limited by the concrete compressive stress at midspan at the intermediate stage due to sustained loads, followed by the concrete tensile stress at midspan at the final stage due to total loads.
f(t) T L & f(c) T D L	The number of strands is simultaneously limited by the concrete tensile stress at midspan at the final stage due to total loads and the compressive stress at midspan at the intermediate stage due to sustained loads.

7.3 CONTROLLING LIMIT STATES FOR AASHTO STANDARD AND LRFD SPECIFICATIONS

7.3.1 AASHTO Standard Specifications

Tables D.1 through D.12 of Appendix D provide controlling limit states for spans from 90 ft. to maximum span lengths at 10 ft. intervals for different concrete classes and spacing of Type IV beams designed using the AASHTO Standard Specifications (AASHTO 2002a). Tables 7.3 through 7.4 show the controlling limit states for maximum span lengths, together with required number of strands and concrete release strengths, for different concrete classes and girder spacings. Both 0.5 in. and 0.6 in. diameter strands were considered.

Table 7.3. Summary of Controlling Limit States and Maximum Spans (AASHTO Standard Specifications, Strand Diameter = 0.5 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	4826	4.25	126.5	42	f(c) Total Dead Load
	5077	5.00	123.4	44	f(c) Total Dead Load
	5328	5.75	120.5	46	f(c) Total Dead Load
	5236	7.00	115.6	48	f(c) Total Dead Load
	5704	8.50	110.6	52	f(c) Total Dead Load
	5941	9.00	109.1	54	f(c) Total Dead Load*
8000	6005	4.25	141.8	56	f(c) Total Dead Load
	6214	5.00	137.9	58	f(c) Total Dead Load
	6388	5.75	134.0	62	f(c) Total Dead Load
	6556	7.00	128.0	66	f(c) Total Dead Load
	6565	8.50	121.2	70	f(c) Total Dead Load
	6645	9.00	119.1	70	f(t) T L / f(c)_T D L
10000	7500	4.25	152.6	70	f(c) Total Dead Load
	7500	5.00	146.7	72	f(t) T L / f(c) T D L
	7500	5.75	141.7	76	f(t) T L / f(c) T D L
	7500	7.00	133.1	76	f(t) Total Load [e]
	7500	8.50	124.2	76	f(t) Total Load [e]
	7500	9.00	121.9	78	f(t) Total Load [e]
12000	9000	4.25	155.5	76	f(t) Total Load [e]
	9000	5.00	148.9	76	f(t) Total Load [e]
	9000	5.75	142.9	76	f(t) Total Load [e]
	9000	7.00	134.2	76	f(t) Total Load [e]
	9000	8.50	125.6	78	f(t) Total Load [e]
	9000	9.00	123.0	78	f(t) Total Load [e]

Table 7.4. Summary of Controlling Limit States and Maximum Spans (AASHTO Standard Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	4906	4.25	128.8	30	f(c) Total Dead Load
	5289	5.00	125.9	32	f(c) Total Dead Load
	5671	5.75	123.2	34	f(c) Total Dead Load*
	5651	7.00	117.9	34	f(t) Total Load*
	5622	8.50	109.8	34	f(t) Total Load*
	6000	9.00	109.8	36	f(t) Total Load*
8000	6385	4.25	145.5	40	f(c) Total Dead Load
	6732	5.00	141.8	42	f(c) Total Dead Load
	7444	5.75	138.6	46	f(c) Total Dead Load
	7307	7.00	132.9	48	f(c) Total Dead Load
	7953	8.50	126.9	52	f(c) Total Dead Load*
	7941	9.00	124.2	52	f(t) Total Load*
10000	8110	4.25	159.2	52	f(c) Total Dead Load
	8340	5.00	154.6	56	f(c) Total Dead Load
	8631	5.75	150.4	58	f(c) Total Dead Load
	9140	7.00	143.4	64	f(c) Total Dead Load
	9320	8.50	135.4	68	f(t) T L / f(c)_T L
	9651	9.00	133.1	70	f(t) T L / f(c)_T L
12000	9251	4.25	169.1	66	f(c) Total Dead Load
	9254	5.00	163.0	70	f(c) Total Dead Load
	9375	5.75	156.8	74	f(c) Total Dead Load
	9451	7.00	147.7	76	f(t) Total Load [e]
	9920	8.50	137.8	76	f(t) Total Load [e]
	10069	9.00	134.9	76	f(t) Total Load [e]

The following trends were observed for designs based on the AASHTO Standard Specifications. For shorter spans (90 ft. and, in some cases, 100 ft.), the number of strands required is controlled by the required flexural strength. In some cases, it was necessary to increase the number of strands to provide the required flexural strength.

The number of strands required for longer spans, except the maximum span lengths, is controlled by the concrete tensile stress at midspan for the final load stage. Maximum span lengths are controlled by the concrete compressive stresses due to total dead loads (not including live loads), except when additional prestressing strands cannot be used because an effective eccentricity is reached. In general, an effective eccentricity is reached when 76 or 78 prestressing strands are used, such that additional strands beyond this number do not provide a significant gain in span length. In this case, maximum spans are controlled by the concrete tensile stress under service loads and at midspan. Other exceptions were noted for wider girder spacing (7 ft. or greater) where maximum spans were limited because the number of strands that could be used was controlled by the compressive stress at the beam ends at release.

The concrete strength at release (f'_{ci}) is critical for AASHTO Standard designs for the widest girder spacing (9 ft.) and the lowest concrete strength (6000 psi) when 0.5 in. diameter strands are used, and for wider girder spacing (greater than or equal to 7 ft.) and concrete strengths up to 8000 psi when 0.6 in. diameter strands are used. The stress limits at release were taken as $0.6\,f'_{ci}$ for compression and $7.5\sqrt{f'_{ci}}$ for tension, where f'_{ci} is in psi units (see Section 5.4).

For Type IV beams with the widest girder spacing (9 ft.) and using 0.5 in. diameter strands, the allowable compressive stress at the beam ends during transfer controls the number of strands used for the maximum span length for an f'_c of 6000 psi. When f'_c is 8000 psi, maximum span lengths for wider girder spacings (greater than 7 ft.) are controlled by the compressive stress. When f'_c is 10000 or 12000 psi, maximum span lengths for wider girder spacings are controlled by the maximum number of strands that the Type IV beam can use (once the effective eccentricity is reached). In other words, because maximum span lengths are controlled by the maximum number of strands that produces a gain in length, no increase in maximum span lengths for wider girder spacings (in this case from 5.75 to 9 ft.) using concrete strengths of more than 10000 psi is possible. Moreover, increasing f'_c to be more than 12000 psi does not lead to increases in the maximum span lengths for any of the girder spacings considered (see Table 7.3).

For Type IV beams with wider girder spacing (generally greater than 7 ft.) and using 0.6 in. diameter strands, the allowable compressive stress at the beam ends during transfer controls the number of strands used for maximum span lengths for f'_c values of 6000 and 8000 psi. When f'_c is 10000 psi, maximum span lengths for wider girder spacings (greater than 7 ft.) are controlled by the compressive stress. When f'_c is 12000 psi, maximum span lengths for wider girder spacings are controlled by the maximum number of strands that the Type IV beam can accommodate. Longer maximum span lengths are possible by adjusting the number of 0.6 in. diameter strands for Type IV beams with girder spacings less than or equal to 7 ft. with an f'_c greater than 12000 psi (see Table 7.4).

7.3.2 AASHTO LRFD Specifications

Tables D.13 through D.24 of Appendix D provide controlling limit states for spans from 90 ft. to maximum span lengths at 10 ft. intervals for different concrete strengths and spacings for Type IV beams designed using the AASHTO LRFD Specifications (AASHTO 2002b). Tables 7.5 and 7.6 show the controlling limit states for the maximum span lengths, together with the required number of strands and concrete release strengths.

Like designs under the Standard Specifications, the number of strands required for shorter spans (90 ft. and, in some cases, 100 ft.) is controlled by the flexural moment strength. In some cases, it was necessary to increase the number of strands to provide the required flexural moment strength.

Table 7.5. Summary of Controlling Limit States and Maximum Spans (AASHTO LRFD Specifications, Strand Diameter = 0.5 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	5950	4.25	132.9	56	f(c) Total Dead Load
	5986	5.00	127.7	54	f(t) Total Load*
	5920	5.75	124.5	56	f(t) Total Load*
	5915	7.00	117.8	56	f(t) _Total Load*
	5869	8.50	111.0	56	f(t) _Total Load*
	5862	9.00	108.9	56	f(t) _Total Load*
8000	6731	4.25	144.1	74	f(t) T L / f(c) T D L
	6796	5.00	139.2	76	f(t) T L / f(c)_T D L
	6777	5.75	134.4	76	f(c) Total Dead Load
	6983	7.00	127.9	80	f(c) T D L & f(t) T L
	6990	8.50	120.3	78	f(t) Total Load [e]
	7087	9.00	118.2	78	f(t) Total Load [e]
10000	7500	4.25	145.3	76	f(t) Total Load [e]
	7500	5.00	140.2	78	f(t) Total Load [e]
	7500	5.75	135.4	78	f(t) Total Load [e]
	7500	7.00	128.2	78	f(t) Total Load [e]
	7500	8.50	120.9	78	f(t) Total Load [e]
	7500	9.00	118.7	78	f(t) Total Load [e]
12000	9000	4.25	146.6	78	f(t) Total Load [e]
	9000	5.00	141.1	78	f(t) _Total Load [e]
	9000	5.75	136.2	78	f(t) _Total Load [e]
	9000	7.00	129.1	78	f(t) _Total Load [e]
	9000	8.50	121.7	78	f(t) _Total Load [e]
	9000	9.00	119.5	78	f(t) _Total Load [e]

Table 7.6. Summary of Controlling Limit States and Maximum Spans (AASHTO LRFD Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	f'_{ci} (psi)	Girder Spacing (ft.)	Max. Span (ft.)	No. Strands	Controlling Limit State
6000	5680	4.25	129.5	34	f(t) Total Load*
	5930	5.00	129.6	38	f(t) Total Load*
	5979	5.75	125.0	38	f(t) Total Load*
	5883	7.00	118.2	38	f(t) Total Load*
	5986	8.50	111,3	38	f(t) Total Load*
	5849	9.00	109.2	38	f(t) Total Load*
8000	7682	4.25	147.8	50	f(t) Total Load*
	7994	5.00	143.8	52	f(t) Total Load*
	7967	5.75	138.6	52	f(t) Total Load*
	7930	7.00	131.1	52	f(t) Total Load*
	7939	8.50	123.4	52	f(t) Total Load*
	7884	9.00	121.0	52	f(t) Total Load*
10000	9335	4.25	160.8	74	f(t) T L / f(c) T D L
	9421	5.00	155.0	76	f(t) T L / f(c) T D L
	9393	5.75	149.4	76	f(t) Total Load [e]
	9630	7.00	141.5	76	f(t) Total Load [e]
	9916	8.50	133.0	74	f(t) Total Load*
	9888	9.00	130.1	72	f(t) Total Load*
12000	9590	4.25	161.6	76	f(t) Total Load [e]
	9423	5.00	155.4	76	f(t) Total Load [e]
	9395	5.75	149.8	76	f(t) Total Load [e]
	9613	7.00	141.8	76	f(t) Total Load [e]
	10028	8.50	133.7	76	f(t) Total Load [e]
	10148	9.00	131.2	76	f(t) Total Load [e]

The number of strands required for longer spans, except the maximum span lengths, is controlled by the concrete tensile stress under service loads at midspan. Maximum span lengths are controlled by concrete compressive stresses due to total dead loads (not including live loads), except when additional prestressing strands cannot be used because an effective eccentricity is reached. In general, an effective eccentricity is reached when 76 or 78 prestressing strands are used. Additional strands beyond this number do not provide a significant gain in length because the addition of strands actually results in a reduction in the eccentricity. In this case, maximum spans are controlled by the concrete tensile stress at midspan under service loads. Other exceptions were noted for wider girder spacing (typically greater than 7 ft.), where maximum spans were limited because the number of strands that could be used was controlled by the tensile or compressive stress at the beam ends at release.

The concrete strength at release (f'_{ci}) is critical for LRFD designs for all girder spacings and concrete strength of 6000 psi when 0.5 in. diameter strands are used and for all girder spacings and concrete strengths up to 10000 psi when 0.6 in. diameter strands are used. The stress limits at release were taken as $0.6 f'_{ci}$ for compression and $6.96\sqrt{f'_{ci}}$ for tension, where f'_{ci} is in psi units (see Section 5.4). The significant load demands of LRFD designs using all girder spacings require a large number of prestressing strands for service conditions. This corresponds to high initial prestressing forces at the beam ends during release. The higher the initial prestressing forces, the greater the required initial concrete strengths. Consequently, the initial stresses control because they become even more critical than the final stresses. In this case, there is a need for a high early concrete strength because the optimal amount of time prior to transfer for production purposes is approximately 12 to 24 hours. The strength gain after release is not as critical in these cases.

For LRFD designs using Type IV beams, several maximum span lengths with all girder spacings were controlled by the allowable release compressive stress. However, for LRFD designs for U54 Beams, only the wider girder spacings were controlled by the allowable tensile and compressive release stresses. For Type IV beams with all girder spacings except 4.25 ft. and using 0.5 in. diameter strands, the allowable compressive stress at the beam ends at release controls the number of strands used for the maximum span length for the lowest f'_c of 6000 psi.

When f'_c is greater than or equal to 8000 psi, the maximum span lengths for wider girder spacings (greater than 7 ft.) are controlled by the maximum number of strands (76 or 78) that the Type IV beam can accommodate. Additional strands beyond this number do not provide a significant gain in length. Moreover, longer maximum span lengths are not possible for Type IV beams with any girder spacing for f'_c values greater than 8000 psi (see Table 7.5 and Figure 7.3).

For Type IV beams with 0.6 in. diameter strands, the allowable compressive stress at the beam ends during transfer controls the number of strands used for maximum span length for f'_c values up to 8000 psi for all girder spacings. When f'_c is 10000 psi, maximum span lengths for wider girder spacings (greater than 7 ft.) are controlled by the compressive stress at the beam end during transfer, with a number of strands close to the maximum number because the effective eccentricity is used. When f'_c is 12000 psi, maximum span lengths for all girder spacings are controlled by the maximum number of strands (76 or 78) that the Type IV beam can accommodate, such that additional strands beyond this number do not provide a significant gain in length. Moreover, researchers concluded that longer maximum span lengths are not possible for Type IV beams with any of the girder spacings studied for concrete strengths more than 10000 psi (see Table 7.6 and Figure 7.4).

These trends indicate that for LRFD designs, the concrete strength at release is critical for several maximum span lengths and all girder spacings, where the allowable stress limits at release were taken as $0.6 f'_{ci}$ for compression and $6.96\sqrt{f'_{ci}}$ for tension (see Section 5.4). The significant live load demands for the LRFD designs result in large initial prestressing forces at the beam ends during release. As the initial prestressing force increases, the required initial concrete strengths must also increase. Consequently, initial stresses control because they become even more critical than the final stresses. In this case, there is a need for a high early concrete strength because the optimal amount of time prior to transfer is typically between 12 to 24 hours for production. The strength gain after release is not the most critical factor in these cases.

7.4 STRAND DIAMETER AND CONCRETE STRENGTH

7.4.1 General

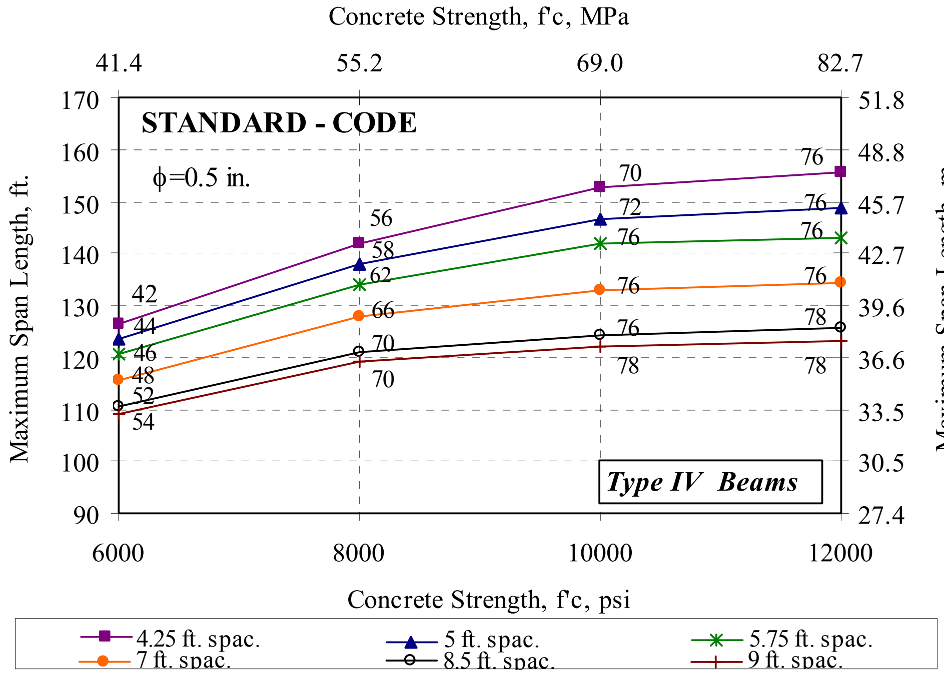
One purpose of the parametric study was to determine the increase in span length possible through the use of different concrete classes. However, the effective use of concrete depends on the diameter of the strands; therefore, the impact of strand diameter was also studied. Figures 7.1 and 7.2 show the trends for maximum span lengths versus various concrete strengths for each girder spacing considered. These graphs correspond to strand diameters of 0.5 and 0.6 in., with designs following both the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b). These graphs help to describe how strand diameter impacts the effective use of concrete strength and, consequently, the maximum span lengths that can be obtained. The maximum span based on the flexural design nearly levels off beyond a certain concrete strength. This leveling off occurs when additional prestressing strands (no more than 76 or 78) do not produce any gain in length.

7.4.2 Trends Observed for AASHTO Standard Specifications

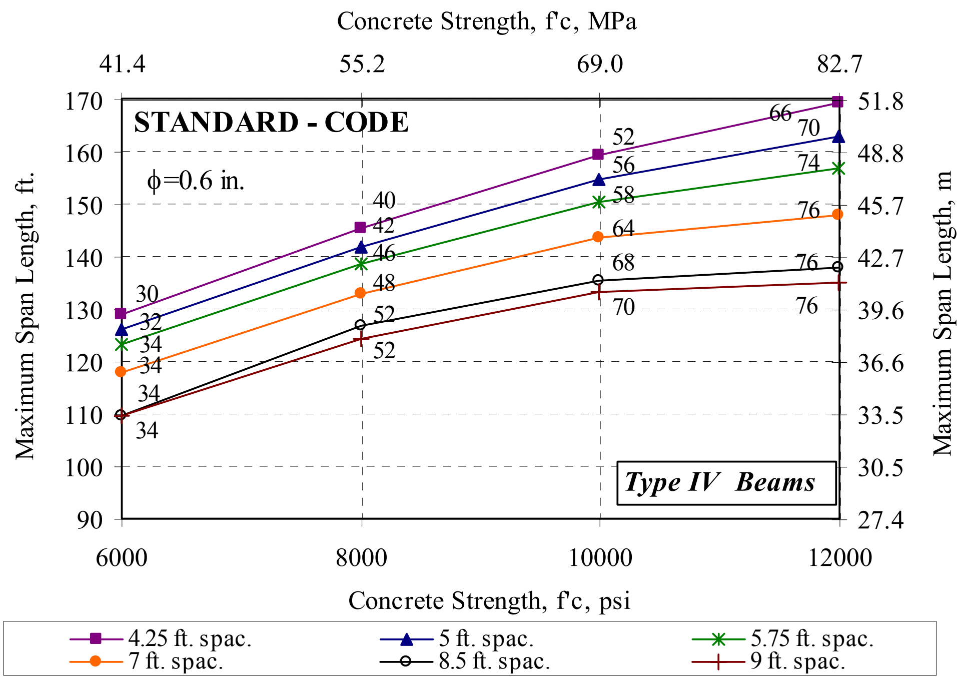
Figure 7.1a shows that the Type IV beam with 0.5 in. diameter strands designed under the AASHTO Standard Specifications can fully utilize concrete strengths up to 10000 psi. The maximum span lengths nearly level off at this strength. Figure 7.1b shows that the Type IV beam with 0.6 in. diameter strands can fully utilize concrete compressive strengths up to 12000 psi and beyond in some cases (12000 psi was the maximum strength considered).

7.4.3 Trends Observed for AASHTO LRFD Specifications

Figure 7.2a shows that the Type IV beam with 0.5 in. diameter strands designed under the AASHTO LRFD Specifications can fully utilize concrete compressive strengths up to 8000 psi. The maximum span lengths nearly level off at this strength. Figure 7.2b shows that the Type IV beam with 0.6 in. diameter strands designed under the AASHTO LRFD Specifications can fully utilize concrete compressive strengths up to 10000 psi. Table 7.7 summarizes these trends.

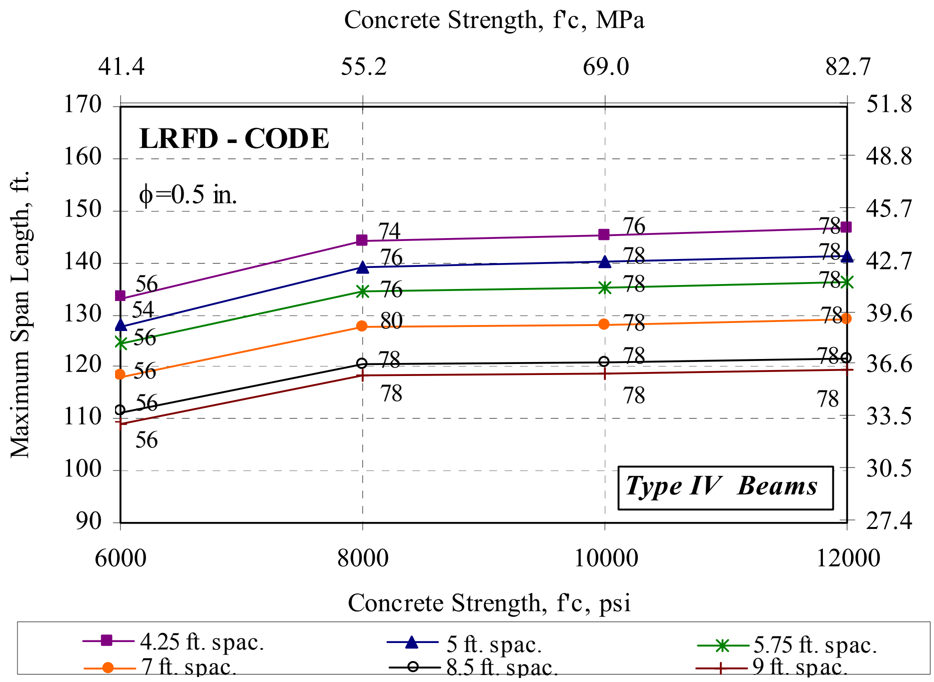


(a) Strand Diameter = 0.5 in.

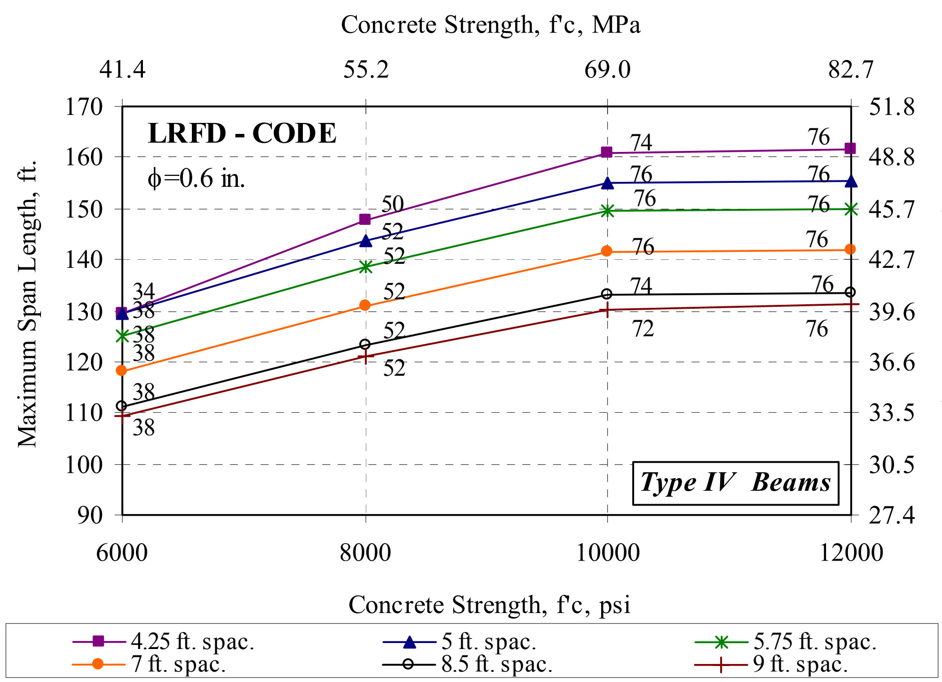


(b) Strand Diameter = 0.6 in.

Figure 7.1. AASHTO Standard Specifications – Maximum Span Length versus Concrete Strength for Type IV Girders.



(a) Strand Diameter = 0.5 in.



(b) Strand Diameter = 0.6 in.

Figure 7.2. AASHTO LRFD Specifications – Maximum Span Length versus Concrete Strength for Type IV Girders.

Table 7.7. Effective Concrete Strength (Type IV Beams).

Strand Diameter (in.)	Girder Spacing (ft.)	Effective Concrete Strength at Maximum Span Length (psi)	
		Standard	LRFD
0.5	All	10000	8000
0.6	All	12000	10000

7.4.4 Impact of Strand Diameter on Maximum Spans

Larger prestressing forces are possible to fully utilize HSC when 0.6 in. diameter strands are used. A comparison of maximum achievable spans for 0.5 and 0.6 in. strand diameters is shown in Tables 7.8 and 7.9 for the Standard and LRFD Specifications, respectively.

Table 7.8. Maximum Spans for 0.5 in. and 0.6 in. Diameter Strands (AASHTO Standard Specifications).

f'_c (psi)	Girder Spacing (ft.)	Maximum Span Length (ft.)		Difference ft. (%)
		Strand Diameter = 0.5 in.	Strand Diameter = 0.6 in.	
6000	4.25	126.5	128.8	2.3 (1.8)
	5.00	123.4	125.9	2.5 (2.0)
	5.75	120.5	123.2	2.7 (2.3)
	7.00	115.6	117.9	2.3 (1.9)
	8.50	110.6	109.8	-0.8 (-0.8)
	9.00	109.1	109.8	0.6 (0.6)
8000	4.25	141.8	145.5	3.7 (2.6)
	5.00	137.9	141.8	3.9 (2.8)
	5.75	134.0	138.6	4.6 (3.4)
	7.00	128.0	132.9	4.9 (3.8)
	8.50	121.2	126.9	5.8 (4.7)
	9.00	119.1	124.2	5.0 (4.2)
10000	4.25	152.6	159.2	6.6 (4.3)
	5.00	146.7	154.6	7.9 (5.4)
	5.75	141.7	150.4	8.7 (6.1)
	7.00	133.1	143.4	10.4 (7.8)
	8.50	124.2	135.4	11.2 (9.0)
	9.00	121.9	133.1	11.2 (9.2)
12000	4.25	155.5	169.1	13.6 (8.8)
	5.00	148.9	163.0	14.2 (9.5)
	5.75	142.9	156.8	13.9 (9.7)
	7.00	134.2	147.7	13.5 (10.1)
	8.50	125.6	137.8	12.2 (9.7)
	9.00	123.00	134.9	11.9 (9.7)

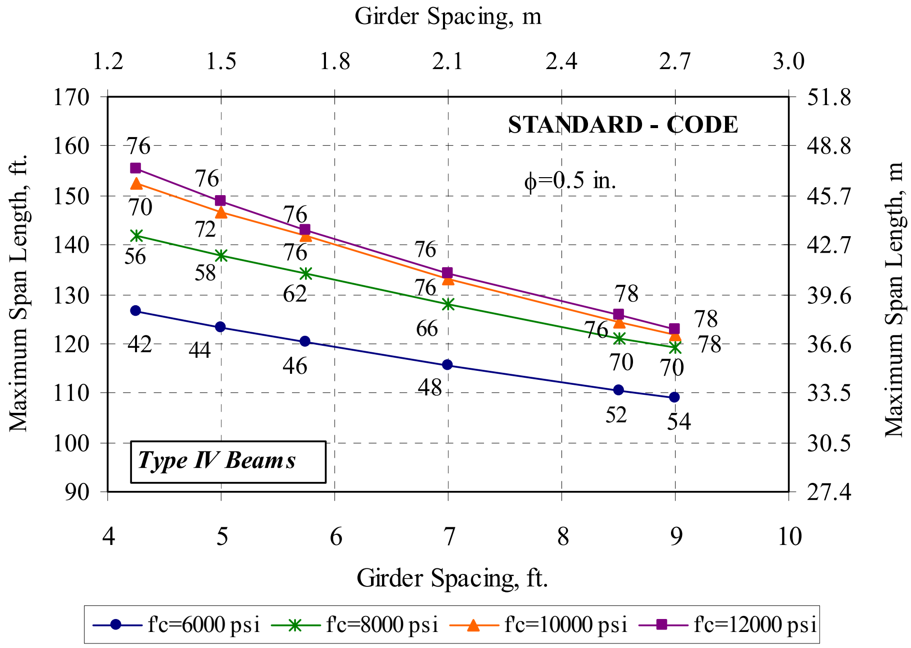
Table 7.9. Maximum Spans for 0.5 in. and 0.6 in. Diameter Strands (AASHTO LRFD Specifications).

f'_c (psi)	Girder Spacing (ft.)	Maximum Span Length (ft.) Strand Diameter = 0.5 in.	Maximum Span Length (ft.) Strand Diameter = 0.6 in.	Difference ft. (%)
6000	4.25	132.9	129.5	-3.3 (-2.4)
	5.00	127.7	129.6	1.9 (1.5)
	5.75	124.5	125.0	0.5 (0.4)
	7.00	117.8	118.2	0.3 (0.3)
	8.50	111.0	111.3	0.3 (0.3)
	9.00	108.9	109.2	0.2 (0.2)
8000	4.25	144.1	147.8	3.7 (2.6)
	5.00	139.2	143.8	4.6 (3.3)
	5.75	134.3	138.6	4.3 (3.2)
	7.00	127.9	131.1	3.2 (2.5)
	8.50	120.3	123.4	3.0 (2.5)
	9.00	118.2	121.0	2.9 (2.4)
10000	4.25	145.3	160.8	15.5 (10.7)
	5.00	140.2	155.0	14.8 (10.5)
	5.75	135.4	149.4	14.1 (10.4)
	7.00	128.2	141.5	13.3 (10.3)
	8.50	120.9	133.0	12.1 (10.0)
	9.00	118.7	130.1	11.4 (9.6)
12000	4.25	146.6	161.6	15.0 (10.3)
	5.00	141.1	155.4	14.2 (10.1)
	5.75	136.2	149.8	13.6 (10.0)
	7.00	129.1	141.8	12.7 (9.8)
	8.50	121.7	133.6	11.9 (9.8)
	9.00	119.5	131.2	11.7 (9.8)

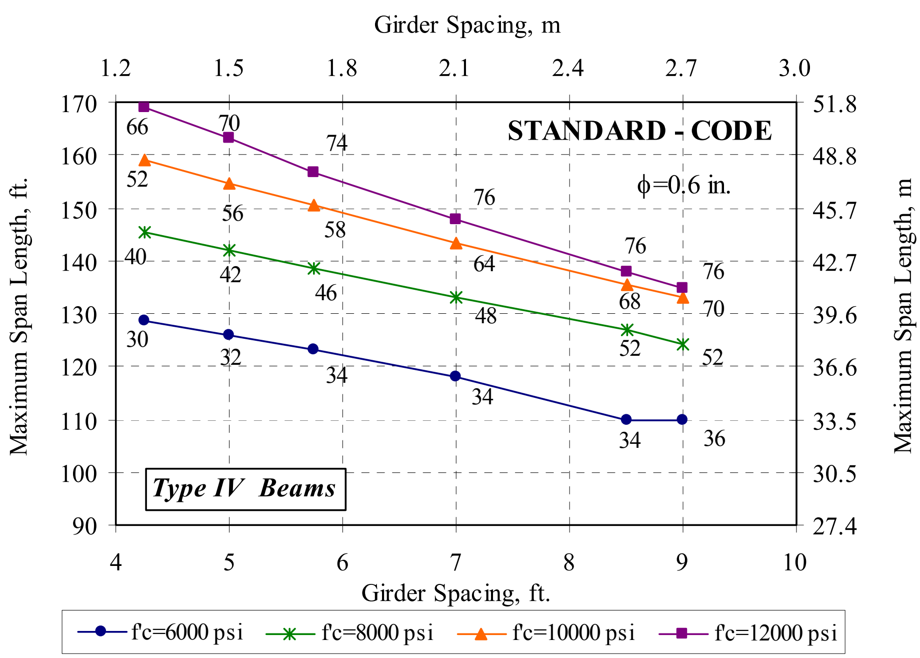
In general, the results show an increase in maximum spans for a given f'_c and girder spacing when 0.6 in. diameter strands are used versus 0.5 in. diameter strands. Percentage increases in maximum span to about 10 percent were found when using 0.6 in. diameter strands for both specifications. As an exception, Table 7.9 shows a percentage decrease of 2.4 percent for maximum spans with the smallest girder spacing (4.25 ft.) designed for an f'_c of 6000 psi using the LRFD Specifications.

7.4.5 Impact of Increasing Concrete Compressive Strengths

Figures 7.3 and 7.4 show plots of maximum span length versus girder spacing for various concrete compressive strengths. The numerical values adjacent to each data point indicate the required number of strands for the corresponding set of design parameters. These graphs show the benefits of higher f'_c values in terms of increased maximum spans. Table 7.10 shows the percentage increase in maximum span when raising f'_c from 6000 psi to the maximum effective strength.

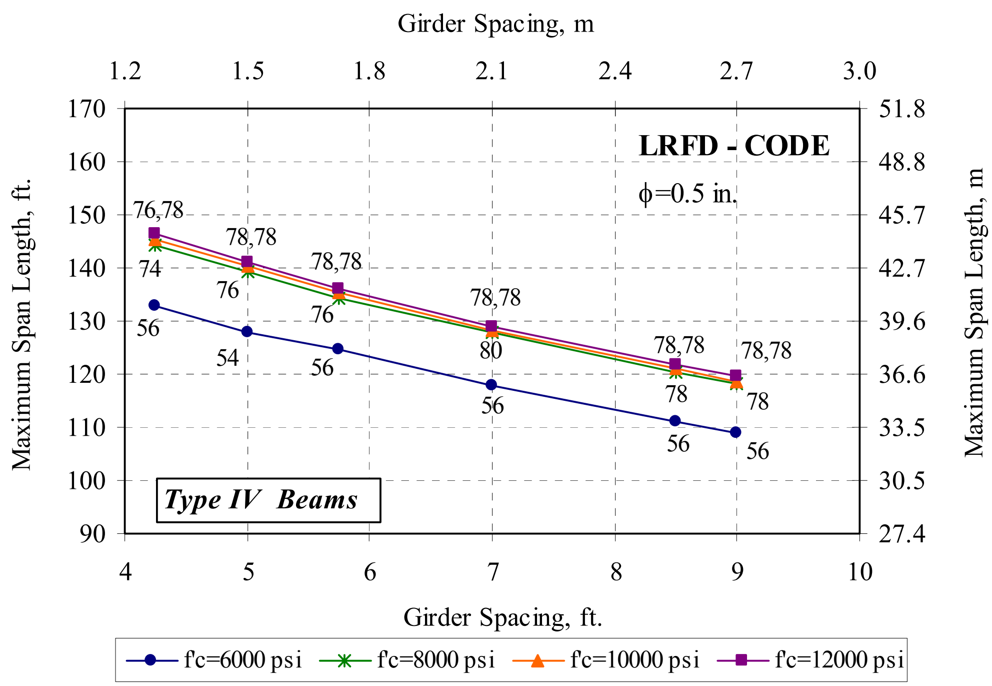


(a) Strand Diameter = 0.5 in.

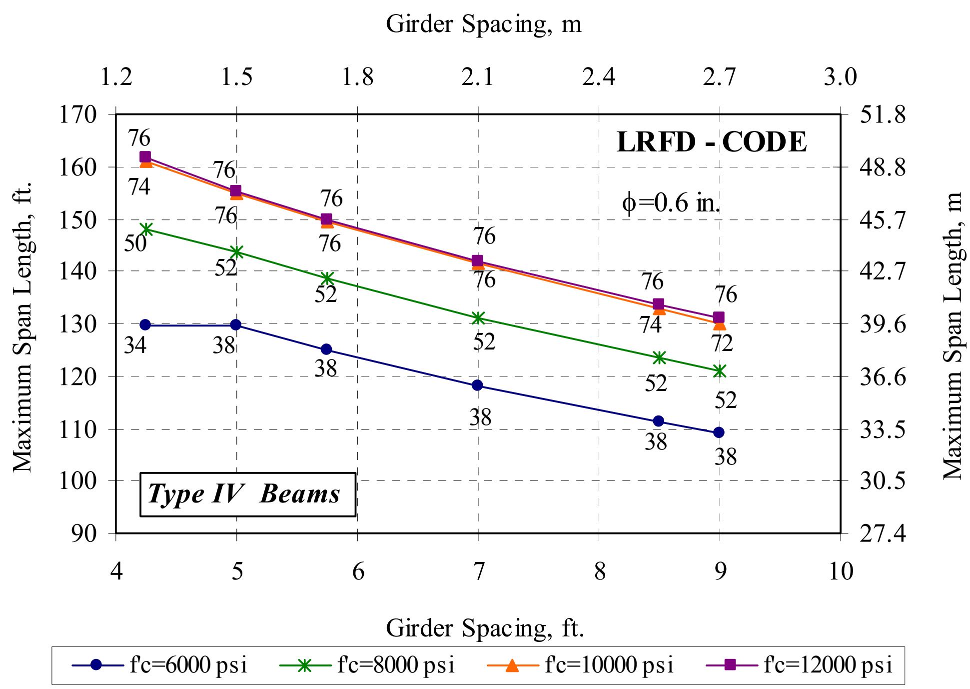


(b) Strand Diameter = 0.6 in.

Figure 7.3. AASHTO Standard Specifications – Maximum Span Length versus Girder Spacing for Type IV Girders.



(a) Strand Diameter = 0.5 in.



(b) Strand Diameter = 0.6 in.

Figure 7.4. AASHTO LRFD Specifications – Maximum Span Length versus Girder Spacing for Type IV Girders.

Table 7.10. Impact of Increasing Concrete Compressive Strengths.

Strand Diameter (in.)	Girder Spacing (ft.)	Average Increase in Max. Span Length ft. (%)		Effective Range of Concrete Strength (psi)	
		Standard	LRFD	Standard	LRFD
0.5	All	19 (16)	10 (9)	6000 – 10000	6000 – 8000
0.6	All	32 (27)	20 (20)	6000 – 12000	6000 – 10000

7.5 COMPARISON OF AASHTO STANDARD AND LRFD SPECIFICATIONS

7.5.1 Comparison of Controlling Limit States

Trends for the controlling limit states for shorter, longer, and maximum span lengths are summarized below. Tables 7.11 and 7.12 show comparisons of controlling limit states for maximum spans. For both specifications, most of the shorter spans are controlled by the required nominal flexural strength, while longer span lengths are controlled by the tensile stresses due to the total load at midspan. The controlling limit states for maximum span lengths are different for designs using the Standard and LRFD Specifications. In general, maximum spans are controlled by the compressive stress due to the total dead loads (sustained loads) whether they are designed under the Standard or LRFD Specifications. The exception is when the allowable tensile stress under total loads would be exceeded because no additional prestressing strands can be used with the Type IV beam section or because the stresses at the beam ends during transfer initially limit the number of strands.

Designs using concrete strengths beyond the effective concrete strength do not provide a significant gain in length because no additional strands can be used with the Type IV beam section. Therefore, designs using strengths beyond the effective strengths are not taken into account in establishing these trends (see Table 7.7).

Table 7.11. Comparison of Limit States That Control Maximum Spans for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.5 in.).

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit State	
		Standard	LRFD
6000	4.25	f(c)_Total Dead Load	f(c) _Total Dead Load
	5.00	f(c)_Total Dead Load	f(t) _Total Load*
	5.75	f(c)_Total Dead Load	f(t) _Total Load*
	7.00	f(c)_Total Dead Load	f(t) _Total Load*
	8.50	f(c)_Total Dead Load	f(t) _Total Load*
	9.00	f(c)_Total Dead Load*	f(t) _Total Load*
8000	4.25	f(c)_Total Dead Load	f(t) _T L / f(c)_T D L
	5.00	f(c)_Total Dead Load	f(t) _T L / f(c)_T D L
	5.75	f(c)_Total Dead Load	f(c) _Total Dead Load
	7.00	f(c)_Total Dead Load	f(c) _T D L & f(t)_T L
	8.50	f(c)_Total Dead Load	f(t) _Total Load [e]
	9.00	f(t) _T L / f(c)_T D L	f(t) _Total Load [e]
10000	4.25	f(c)_Total Dead Load	f(t) _Total Load [e]
	5.00	f(t) _T L / f(c)_T D L	f(t) _Total Load [e]
	5.75	f(t) _T L / f(c)_T D L	f(t) _Total Load [e]
	7.00	f(t) _Total Load [e]	f(t) _Total Load [e]
	8.50	f(t) _Total Load [e]	f(t) _Total Load [e]
	9.00	f(t) _Total Load [e]	f(t) _Total Load [e]
12000	4.25	f(t) _Total Load [e]	f(t) _Total Load [e]
	5.00	f(t) _Total Load [e]	f(t) _Total Load [e]
	5.75	f(t) _Total Load [e]	f(t) _Total Load [e]
	7.00	f(t) _Total Load [e]	f(t) _Total Load [e]
	8.50	f(t) _Total Load [e]	f(t) _Total Load [e]
	9.00	f(t) _Total Load [e]	f(t) _Total Load [e]

Table 7.12. Comparison of Limit States That Control Maximum Spans for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit State	
		Standard	LRFD
6000	4.25	f(c) _Total Dead Load	f(t) _Total Load*
	5.00	f(c) _Total Dead Load	f(t) _Total Load*
	5.75	f(c) _Total Dead Load*	f(t) _Total Load*
	7.00	f(t) _Total Dead Load*	f(t) _Total Load*
	8.50	f(t) _Total Load*	f(t) _Total Load*
	9.00	f(t) _Total Load*	f(t) _Total Load*
8000	4.25	f(c) _Total Dead Load	f(t) _Total Load*
	5.00	f(c) _Total Dead Load	f(t) _Total Load*
	5.75	f(c) _Total Dead Load	f(t) _Total Load*
	7.00	f(c) _Total Dead Load	f(t) _Total Load*
	8.50	f(c) _Total Dead Load*	f(t) _Total Load*
	9.00	f(t) _Total Load*	f(t) _Total Load*
10000	4.25	f(c) _Total Dead Load	f(t) _T L / f(c)_T D L
	5.00	f(c) _Total Dead Load	f(t) _T L / f(c)_T DL
	5.75	f(c) _Total Dead Load	f(t) _Total Load [e]
	7.00	f(c) _Total Dead Load	f(t) _Total Load [e]
	8.50	f(t) _T L / f(c)_T D L	f(t) _Total Load*
	9.00	f(t) _T L / f(c)_T D L	f(t) _Total Load*
12000	4.25	f(c) _Total Dead Load	f(t) _Total Load [e]
	5.00	f(c) _Total Dead Load	f(t) _Total Load [e]
	5.75	f(c) _Total Dead Load	f(t) _Total Load [e]
	7.00	f(t) _Total Load [e]	f(t) _Total Load [e]
	8.50	f(t) _Total Load [e]	f(t) _Total Load [e]
	9.00	f(t) _Total Load [e]	f(t) _Total Load [e]

Unlike the U54 beam designs, the compressive stresses at the beam ends during transfer control some of the maximum spans for Type IV beams with wider spacings designed using the Standard Specifications. Like the U54 beam designs, several maximum spans are controlled by the compressive stresses at transfer when they are designed under the LRFD Specifications. However, in this case, not just maximum spans with wider girder spacings are affected rather maximum span lengths for all girder spacings are limited by the compressive stress at transfer (except for one case, see Table 7.5).

7.5.2 Comparison of Maximum Span Lengths

7.5.2.1 General

Tables 7.13 and 7.14 show a comparison of maximum span lengths for designs according to the AASHTO Standard and LRFD Specifications, using 0.5 and 0.6 in. diameter strands, respectively. The tables also provide the required number of strands for each design. The difference in the maximum span length for the LRFD designs is expressed as a percentage change relative to the designs for the Standard Specifications. Table 7.15 provides values for the largest differences in the maximum span lengths for f'_c values up to those strengths that work effectively with the Type IV beam under both codes. Increases in maximum spans are differentiated for girder spacings less than or equal to 5.75 ft. and for girder spacings more than 5.75 ft.

7.5.2.2 0.5 in. Diameter Strands

For designs using 0.5 in. diameter strands, girder spacing less than or equal to 5.75 ft., and f'_c values between 6000 and 8000 psi (range where concrete strength works efficiently with the Type IV beam under both codes), LRFD designs result in increases up to 6.4 ft. (5 percent) in maximum span lengths. Note that for the maximum effective concrete strength of 8000 psi, the maximum increase in maximum span lengths is limited to 2.3 ft. (1.6 percent). For girder spacings more than 5.75 ft. and for f'_c in the range of 6000 to 8000 psi, LRFD designs result in a small decrease of up to 0.9 ft. (0.8 percent). in maximum spans lengths.

Table 7.13. Comparison of Maximum Span Lengths for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.5 in.).

f'_c (psi)	Girder Spacing (ft.)	Standard		LRFD		Difference ft. (%)
		Max. Span (ft.)	**No.**	**Max. Span** (ft.)	**No. Strands**	
6000	4.25	126.5	42	132.9	56	6.4 (5.0)
	5.00	123.4	44	127.7	54	4.3 (3.5)
	5.75	120.5	46	124.5	56	4.0 (3.4)
	7.00	115.6	48	117.8	56	2.2 (1.9)
	8.50	110.6	52	111.0	56	0.4 (0.3)
	9.00	109.1	54	108.9	56	-0.2 (-0.2)
8000	4.25	141.8	56	144.1	74	2.3 (1.6)
	5.00	137.9	58	139.2	76	1.3 (0.9)
	5.75	134.0	62	134.3	76	0.3 (0.2)
	7.00	128.0	66	127.9	80	-0.1 (-0.1)
	8.50	121.2	70	120.3	78	-0.8 (-0.7)
	9.00	119.1	70	118.2	78	-0.9 (-0.8)
10000	4.25	152.6	70	145.3	78	-7.3 (-4.8)
	5.00	146.7	72	140.2	78	-6.5 (-4.4)
	5.75	141.7	76	135.4	78	-6.4 (-4.5)
	7.00	133.1	76	128.2	78	-4.9 (-3.7)
	8.50	124.2	76	120.9	78	-3.3 (-2.7)
	9.00	121.9	78	118.7	78	-3.2 (-2.6)
12000	4.25	155.5	76	146.6	78	-8.9 (-5.7)
	5.00	148.9	76	141.1	78	-7.8 (-5.2)
	5.75	142.9	76	136.2	78	-6.7 (-4.7)
	7.00	134.2	76	129.1	78	-5.1 (-3.8)
	8.50	125.6	78	121.7	78	-3.9 (-3.1)
	9.00	123.0	78	119.5	78	-3.5 (-2.8)

Table 7.14. Comparison of Maximum Span Lengths for AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	Standard		LRFD		Difference ft. (%)
		Max. Span (ft.)	**No. Strands**	**Max. Span** (ft.)	**No. Strands**	
6000	4.25	128.8	30	129.5	34	0.8 (0.6)
	5.00	125.9	32	129.6	38	3.7 (2.9)
	5.75	123.2	34	125.0	38	1.8 (1.5)
	7.00	117.9	34	118.2	38	0.3 (0.2)
	8.50	109.8	34	111.3	38	1.5 (1.4)
	9.00	109.8	36	109.2	38	-0.6 (-0.5)
8000	4.25	145.5	40	147.8	50	2.3 (1.6)
	5.00	141.8	42	143.8	52	2.0 (1.4)
	5.75	138.6	46	138.6	52	0.0 (0.0)
	7.00	132.9	48	131.1	52	-1.9 (-1.4)
	8.50	126.9	52	123.4	52	-3.6 (-2.8)
	9.00	124.2	52	121.0	52	-3.1 (-2.5)
10000	4.25	159.2	52	160.8	74	1.6 (1.0)
	5.00	154.6	56	155.0	76	0.3 (0.2)
	5.75	150.4	58	149.4	76	-1.0 (-0.6)
	7.00	143.4	64	141.5	76	-2.0 (-1.4)
	8.50	135.4	68	133.0	74	-2.4 (-1.8)
	9.00	133.1	70	130.1	72	-3.0 (-2.2)
12000	4.25	169.1	66	161.6	76	-7.5 (-4.5)
	5.00	163.0	70	155.4	76	-7.7 (-4.7)
	5.75	156.8	74	149.8	76	-7.0 (-4.5)
	7.00	147.7	76	141.8	76	-5.9 (-4.0)
	8.50	137.84	76	133.65	76	-4.2 (-3.0)
	9.00	134.92	76	131.22	76	-3.7 (-2.7)

Table 7.15. Maximum Difference in Maximum Span Length for LRFD Relative to Standard Specifications.

Girder Spacing	Strand Diameter = 0.5 in.				Strand Diameter = 0.6 in.			
	6000 psi	8000 psi	10000 psi	12000 psi	6000 psi	8000 psi	10000 psi	12000 psi
S ≤ 5.75 ft.	6.4 ft. (5.0%)	2.3 ft. (1.6%)	---	---	3.7 ft. (2.9%)	2.3 ft. (1.6%)	1.6 ft. (1.0%)	---
S > 5.75 ft.	-0.2 ft. (-0.2%)	0.9 ft. (-0.8%)	---	---	-0.6 ft. (-0.5%)	-3.6 ft. (-2.8%)	-3.0 ft. (-2.2%)	---

7.5.2.3 0.6 in. Diameter Strands

For designs using 0.6 in. diameter strands with girder spacings less than or equal to 5.75 ft. and concrete strengths in the range of 6000 to 10000 psi (the range where concrete strength works efficiently with the Type IV beam under both codes), LRFD designs result in increases of up to 3.7 ft. (2.9 percent) in maximum span lengths (see Table 7.15). This value varies with concrete strength class and also with girder spacing. Note that for the maximum effective concrete strength of 10000 psi, the largest increase in the maximum span length is 1.6 ft. (1.0 percent).

For girder spacings more than 5.75 ft., a comparison indicates that for concrete strengths from 6000 to 10000 psi, LRFD designs result in 3.6 ft. (2.8 percent) decreases in maximum span lengths. In general, this value varies with concrete strength class and girder spacing. Note that for the range of concrete strength from 8000 to 10000 psi (maximum effective concrete strength), the maximum percentage decreases are almost the same. Shorter maximum spans (up to 3.6 ft.) were obtained under the LRFD Specifications for the cases where maximum span lengths are limited by the number of strands that can be used or by the stresses at the beam ends during transfer.

7.5.3 Comparison of Number of Strands

Tables 7.16 through 7.21 show differences in the number of 0.6 in. diameter strands required for span lengths from 90 ft. to the maximum spans designed under the LRFD and the Standard Specifications. Each table shows the designs for a different girder spacing. The difference in the number of strands for maximum spans is not reported because the number of strands for different maximum span lengths cannot be directly compared. For all girder spacings and for the same span, the LRFD designs required an increase of between 0 and 18 strands compared to designs using the Standard Specifications.

The effect of the 0.8 factor included in LRFD Service III limit state compared with the factor of 1.0 considered in the Standard Specifications should result in a reduction of number of strands required for the same load requirements. However, more strands are needed for all girder spacings considered. Larger LRFD live load demands explain the larger number of strands. Also, for Type IV beams designed under the LRFD Specifications, the same live load distribution factor expression was used for all girder spacings.

Table 7.16. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 4.25 ft.).

f'_c	Standard		LRFD		Difference No. Strands
(psi)	Length (ft.)	No. Strands	Length (ft.)	No. Strands	
6000	90	12	90	14	2
	100	16	100	18	2
	110	20	110	24	4
	120	24	120	28	4
	128.8	30	129.5	34	-
8000	90	12	90	14	2
	100	16	100	18	2
	110	20	110	22	2
	120	24	120	28	4
	130	30	130	34	4
	140	36	140	42	6
	145.5	40	147.8	50	-
10000	90	12	90	14	2
	100	14	100	18	4
	110	18	110	22	4
	120	24	120	28	4
	130	28	130	34	6
	140	36	140	42	6
	150	42	150	52	10
	159.2	52	160	70	-
	-	-	160.8	74	-

Table 7.16. Continued

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	No. Strands	Length (ft.)	No. Strands	
12000	90	12	90	14	2
	100	14	100	18	4
	110	18	110	22	4
	120	22	120	26	4
	130	28	130	34	6
	140	34	140	42	8
	150	42	150	52	10
	160	52	160	70	18
	169.1	66	161.6	76	-

Table 7.17. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 5 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	No. Strands	Length (ft.)	No. Strands	
6000	90	14	90	16	2
	100	18	100	20	2
	110	22	110	26	4
	120	28	120	32	4
	125.9	32	129.6	38	-
8000	90	14	90	16	2
	100	18	100	20	2
	110	22	110	24	2
	120	26	120	32	6
	130	34	130	38	4
	140	42	140	48	6
	141.8	42	143.8	52	-
10000	90	14	90	16	2
	100	16	100	20	4
	110	22	110	24	2
	120	26	120	30	4
	130	32	130	38	6
	140	40	140	48	8
	150	50	150	62	12
	154.6	56	155.0	76	-
12000	90	14	90	16	2
	100	16	100	18	2
	110	20	110	24	4
	120	26	120	30	4
	130	32	130	38	6
	140	40	140	46	6
	150	48	150	60	12
	160	62	155.4	76	-
	163.0	70	-	-	-

Table 7.18. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 5.75 ft.).

f'_c (psi)	Standard		LRFD		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	16	90	18	2
	100	20	100	22	2
	110	24	110	28	4
	120	32	120	34	2
	122.6	32	125.0	38	-
8000	90	14	90	16	2
	100	20	100	22	2
	110	24	110	28	4
	120	30	120	34	4
	130	38	130	44	6
	138.6	46	138.6	52	-
10000	90	14	90	16	2
	100	18	100	20	2
	110	24	110	26	2
	120	30	120	34	4
	130	36	130	42	6
	140	46	140	54	8
	150	58	149.4	76	-
	150.4	58	-	-	-
12000	90	14	90	16	2
	100	18	100	20	2
	110	22	110	26	4
	120	28	120	32	4
	130	36	130	42	6
	140	44	140	54	10
	150	56	149.8	76	-
	156.8	74	-	-	-

Table 7.19. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 7 ft.).

f'c (psi)	**Standard**		**LRFD**		**Difference No. Strands**
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	18	90	20	2
	100	24	100	26	2
	110	30	110	32	2
	118.7	36	118.2	38	-
8000	90	18	90	18	-
	100	22	100	24	2
	110	28	110	32	4
	120	36	120	40	4
	130	46	130	52	6
	132.9	48	131.1	52	-
10000	90	16	90	18	2
	100	22	100	24	2
	110	28	110	30	2
	120	34	120	40	6
	130	44	130	50	6
	140	58	140	70	12
	143.4	64	141.5	76	-
12000	90	16	90	18	2
	100	22	100	24	2
	110	26	110	30	4
	120	34	120	38	4
	130	44	130	50	6
	140	56	140	70	14
	147.7	76	141.8	76	-

Table 7.20. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 8.5 ft.).

f'_c	Standard		LRFD		Difference
(psi)	**Length** (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	**No. Strands**
6000	90	22	90	22	0
	100	28	100	30	2
	112.2	36	109.4	36	-
8000	90	20	90	22	2
	100	26	100	28	2
	110	34	110	38	4
	120	44	120	48	4
	126.9	52	123.4	52	-
10000	90	20	90	22	2
	100	26	100	28	2
	110	34	110	36	2
	120	44	120	48	4
	130	58	130	64	6
	135.4	68	133.0	74	-
12000	90	20	90	20	0
	100	26	100	28	2
	110	32	110	36	4
	120	42	120	46	4
	130	56	130	64	8
	137.8	76	133.6	76	-

Table 7.21. Comparison of Number of Strands – AASHTO Standard and LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 9 ft.).

f'_c	Standard		LRFD		Difference
(psi)	**Length** (ft.)	**No Strands**	**Length** (ft.)	**No Strands**	**No. Strands**
6000	90	22	90	24	2
	100	30	100	30	0
	109.8	36	109.2	38	-
8000	90	22	90	22	0
	100	28	100	30	2
	110	36	110	40	4
	120	48	120	52	4
	124.2	52	121.0	52	-
10000	90	20	90	22	2
	100	28	100	30	2
	110	36	110	38	2
	120	46	120	50	4
	130	62	130	72	10
	133.1	70	130.1	72	-
12000	90	20	90	22	2
	100	26	100	28	2
	110	34	110	38	4
	120	44	120	50	6
	130	60	130	72	12
	134.9	76	131.2	76	-

7.6 STRESSES AT TRANSFER AND TRANSFER LENGTH

Results for the Type IV beam designs indicate that the concrete tensile stress at transfer does not control any maximum spans designed under the Standard or LRFD Specifications. For the parametric study, the allowable tensile stress at release was taken as the highest limit ($7.5\sqrt{f'_{ci}}$ for Standard designs and $6.96\sqrt{f'_{ci}}$ for LRFD designs, where f'_{ci} is in psi units). Researchers selected this criterion to be consistent with the TxDOT design software, PSTRS14 (TxDOT 1980), rather than using the lower limit of the minimum of $3\sqrt{f'_{ci}}$ (where f'_{ci} is in psi units) or 200 psi, provided by both AASHTO Specifications, when no additional bonded reinforcement is used. Therefore, bonded reinforcement is necessary at the beam ends for the designs in this study. The allowable compressive stress at release was not varied, and the limit is consistent with that given in the Standard and LRFD Specifications.

The parametric study uses the same approach as that used in PSTRS14 program, where stresses at the beam ends were determined assuming the strands develop instantaneously after the debonded length. In this case, the strand transfer length is conservatively assumed to be zero. However, the AASHTO Specifications specify that the transfer length is 60 strand diameters.

To assess the impact of this conservative assumption for the transfer length, researchers conducted additional analysis for several critical cases. In addition, the impact of the lower tensile stress limit at release was evaluated. The cases considered were the maximum spans using 0.6 in. diameter strands for both specifications. The allowable tensile stress at transfer specified as the minimum of $3\sqrt{f'_{ci}}$ or 200 psi, results in the use of 200 psi, which seems more appropriate for normal strength concrete (up to 6000 psi). Therefore, a limit of $3\sqrt{f'_{ci}}$ was used in this evaluation because the 200 psi limit would dramatically reduce the span lengths for higher strength concrete.

7.6.1 Impact on the Controlling Limit States

7.6.1.1 Standard Specifications

Table 7.22 shows the impact of the allowable release stresses and transfer length on the controlling limit states for maximum span lengths designed using the AASHTO Standard Specifications and using 0.6 in. diameter strands.

Table 7.22. Controlling Limit States for Maximum Spans for Different Allowable Release Stresses and Transfer Lengths (AASHTO Standard Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit States			
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$		$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$	
		***L*transfer=0** (This Study)	***L*transfer=60 ϕ**	***L*transfer=0**	***L*transfer=60 ϕ**
6000	4.25	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.75	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	7.0	f(t) Total Load*	f(c) Total Dead Load*	f(t) Total Load*	f(t) Total Load*
	8.5	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	9.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
8000	4.25	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.75	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	7.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	8.5	f(c) Total Dead Load*	f(t) T L / f(c) T D L	f(c) Total Dead Load*	f(t) Total Load*
	9.0	f(t) Total Load*	f(c) Total Dead Load*	f(t) Total Load*	f(t) Total Load*
10000	4.25	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.75	f(c) Total Dead Load	f(c) T D L & f(t) TL	f(c) Total Dead Load	f(c) Total Dead Load
	7.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	8.5	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L
	9.0	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L
12000	4.25	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.0	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	5.75	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load	f(c) Total Dead Load
	7.0	f(t) Total Load [e]	f(t) Total Load [e]	f(t) Total Load [e]	f(t) Total Load [e]
	8.5	f(t) Total Load [e]	f(t) Total Load [e]	f(t) Total Load [e]	f(t) Total Load [e]
	9.0	f(t) Total Load [e]	f(t) Total Load [e]	f(t) Total Load [e]	f(t) Total Load [e]

Notes: See Table 7.2 for Limit State Notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

For designs with an allowable tensile stress at transfer of $7.5\sqrt{f'_{ci}}$, no significant differences were found in the controlling limit states for maximum spans when transfer lengths of 0 and 60 strand diameters were used. The transfer length used also had no significant impact on the controlling limit states for designs using an allowable tensile stress at transfer of $3\sqrt{f'_{ci}}$. In particular, for the lower allowable stress at transfer, maximum spans with wider girder spacings designed with concrete strengths up to 8000 psi were controlled by the compressive concrete stress at beam ends during transfer.

7.6.1.2 LRFD Specifications

Table 7.23 shows the impact of the allowable release stress and transfer length on the controlling limit states for maximum spans for designs using the LRFD Specifications and 0.6 in. diameter strands. For designs with the lower tensile stress limit at transfer of $3\sqrt{f'_{ci}}$, no significant differences were found on the controlling limit states for maximum spans when transfer lengths of 0 and 60 strand diameters were used. In particular, maximum spans for all girder spacings designed with f'_c values up to 8000 psi, and maximum spans with wider girder spacings (8.5 ft. and 9 ft.) designed with f'_c values up to 10000 psi were controlled by the compressive concrete stress at beam ends during transfer.

Table 7.23. Controlling Limit States for Maximum Spans for Different Allowable Release Stresses and Transfer Lengths (AASHTO LRFD Specifications, Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit States			
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$		$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$	
		*L*transfer=0 (This Study)	*L*transfer=60 ϕ	*L*transfer=0	*L*transfer=60 ϕ
6000	4.25	f(t) Total Load*	f(c) Total Dead Load*	f(t) Total Load*	f(t) Total Load*
	5.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	5.75	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	7.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	8.5	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	9.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
8000	4.25	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	5.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	5.75	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	7.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	8.5	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	9.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*

Table 7.23. Continued.

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit States			
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$		$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$	
		*L*transfer=0 (This Study)	*L*transfer=60 ϕ	*L*transfer=0	*L*transfer=60 ϕ
10000	4.25	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L	f(t) TL / f(c) T D L	f(t) T L / f(c) T D L
	5.0	f(t) TL / f(c) T D L	f(t) T L / f(c) T D L	f(t) TL / f(c) T D L	f(t) T L / f(c) T D L
	5.75	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	7.0	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	8.5	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
	9.0	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*	f(t) Total Load*
12000	4.25	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	5.0	f(t) Total Load ^{e}	f(c) T D L & f(t) TL	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	5.75	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	7.0	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	8.5	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}
	9.0	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}	f(t) Total Load ^{e}

Notes: See Table 7.2 for Limit State Notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

7.6.2 Impact on Maximum Span Lengths

7.6.2.1 Standard Specifications

Table 7.24 shows the impact of the allowable tensile stress at release and transfer length on maximum span lengths for designs using the Standard Specifications and 0.6 in. diameter strands. For both tensile stress limits at release, the use of a transfer length of 60 strand diameters versus a transfer length of zero does not have an impact on the maximum span lengths, except for three cases using each limit where increases up to 2.3 ft. (2.1 percent) were found for girder spacings greater than 7 ft. and for an f'_c of 6000 psi.

Table 7.24 shows no differences in maximum span lengths when the limit for the tensile stress changes from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with zero transfer length), except for a case (for a girder spacing of 9 ft. and for a concrete strength of 6000 psi) where the decrease in maximum span length was 2.4 ft. (2.2 percent). The same table shows only small differences (less than 1.0 ft.) in maximum span lengths when the limit for the tensile stress changes from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with transfer length of 60 strand diameters). These differences are representative only for the lowest concrete strength (6000 psi).

Table 7.24. Maximum Span Lengths for Different Allowable Release Stresses and Transfer Lengths (AASHTO Standard Specifications, Strand Diameter = 0.6 in.).

		Maximum Span Lengths						
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$			$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6 f'_{ci}$			
		*L*transfer=0	*L*transfer=60 ϕ		*L*transfer=0		*L*transfer=60 ϕ	
f'_c	Girder Spacing (ft.)	Max. Span (This Study) (ft.)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)
6000	4.25	128.8	128.7	-	128.8	-	128.7	-
	5.0	125.9	125.9	-	125.9	-	125.9	-
	5.75	123.2	123.2	-	122.6	-	122.6	-
(psi)	7.0	117.9	118.7	0.8 (0.7)	117.9	-	118.7	0.8 (0.7)
	8.5	109.8	112.1	2.3 (2.1)	109.8	-	109.7	-0.1 (-0.1)
	9.0	109.8	111.6	1.8 (1.6)	107.4	-2.4 (-2.2)	109.7	-0.1 (-0.1)
8000	4.25	145.5	145.5	-	145.5	-	145.5	-
	5.0	141.8	141.8	-	141.8	-	141.8	-
	5.75	138.6	138.6	-	138.6	-	138.6	-
	7.0	132.9	132.9	-	132.9	-	132.9	-
	8.5	126.9	126.8	-0.1 (-0.1)	126.9	-	126.8	-0.1 (-0.1)
	9.0	124.2	125.1	0.9 (0.7)	124.2	-	125.0	0.8 (0.7)
10000	4.25	159.2	159.2	-	159.2	-	159.2	-
	5.0	154.6	154.6	-	154.6	-	154.6	-
	5.75	150.4	150.4	-	150.4	-	150.4	-
	7.0	143.4	143.4	-	143.4	-	143.4	-
	8.5	135.4	135.4	-	135.4	-	135.4	-
	9.0	133.1	133.1	-	133.1	-	133.1	-
12000	4.25	169.1	169.1	-	169.1	-	169.1	-
	5.0	163.0	163.0	-	163.0	-	163.0	-
	5.75	156.8	156.8	-	156.8	-	156.8	-
	7.0	147.7	147.7	-	147.7	-	147.7	-
	8.5	137.8	137.8	-	137.8	-	137.8	-
	9.0	134.9	134.9	-	134.9	-	134.9	-

Notes: See Table 7.2 for Limit State Notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

7.6.2.2 LRFD Specifications

Table 7.25 shows the impact of the allowable tensile stress at release and transfer length on maximum span lengths for designs using the LRFD Specifications and 0.6 in. diameter strands. For designs with an allowable tensile stress at transfer of $7.5\sqrt{f'_{ci}}$, the use of a transfer length of 60 strand diameters resulted in increases of up to 7.1 ft. (5.5 percent) in the maximum span lengths. For designs using an allowable tensile stress at transfer of $3\sqrt{f'_{ci}}$, increases in maximum span lengths up to 3.2 ft. (2.2 percent) were found when the transfer length was changed from 0 to 60 strand diameters. Negligible differences were observed for the maximum span lengths when the limit for the tensile stress changed from $7.5\sqrt{f'_{ci}}$ to $3\sqrt{f'_{ci}}$, with zero transfer length. Increases in maximum span lengths up to 3.2 ft. (2.2 percent) occurred when the limit for the tensile stress was reduced from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with transfer length of 60 strand diameters).

Table 7.25. Maximum Span Lengths for Different Allowable Release Stresses and Transfer Lengths (AASHTO LRFD Specifications, Strand Diameter = 0.6 in.).

		Maximum Span Lengths						
		$f_t = 7.5\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$			$f_t = 3\sqrt{f'_{ci}}$, $f_c = 0.6f'_{ci}$			
		*L*transfer=0	*L*transfer=60 ϕ		*L*transfer=0		*L*transfer=60 ϕ	
f'_c (psi)	Girder Spacing (ft.)	Max. Span (This Study) (ft.)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)	Max. Span (ft.)	Difference ft. (%)
6000	4.25	129.5	136.6	7.1 (5.5)	129.5	-	132.3	2.7 (2.1)
	5.0	129.6	131.8	2.2 (1.7)	129.6	-	131.8	2.2 (1.7)
	5.75	125.0	127.1	2.1 (1.7)	125.0	-	127.1	2.1 (1.7)
	7.0	118.2	120.2	2.0 (1.7)	118.2	-	120.2	2.0 (1.7)
	8.5	111.3	113.1	1.8 (1.6)	111.3	-	113.2	1.9 (1.7)
	9.0	109.2	109.2	-	109.2	-	109.2	-
8000	4.25	147.8	151.1	3.2 (2.2)	147.8	-	151.1	3.2 (2.2)
	5.0	143.8	146.5	2.7 (1.9)	143.8	-	145.2	1.4 (1.0)
	5.75	138.6	140.0	1.4 (1.0)	138.6	-	140.0	1.4 (1.0)
	7.0	131.1	133.5	2.4 (1.8)	131.1	-	132.4	1.3 (1.0)
	8.5	123.4	124.6	1.2 (1.0)	123.4	-	124.6	1.2 (1.0)
	9.0	121.0	122.2	1.2 (1.0)	121.0	-	121.0	-
10000	4.25	160.8	160.7	-0.2 (-0.1)	160.8	-	160.7	-0.1 (-0.1)
	5.0	155.0	154.8	-0.2 (-0.1)	155.0	-	154.8	-0.2 (-0.1)
	5.75	149.4	149.4	-	149.4	-	149.3	-0.1 (-0.1)
	7.0	141.5	141.5	-	141.8	0.3 (0.2)	141.8	0.3 (0.2)
	8.5	133.0	133.0	-	133.0	-	133.0	-
	9.0	130.1	130.1	-	130.1	-	130.1	-
12000	4.25	161.6	161.4	-0.2 (-0.1)	161.6	-	161.4	-0.2 (-0.1)
	5.0	155.4	155.2	-0.2 (-0.1)	155.4	-	155.2	-0.2 (-0.1)
	5.75	149.8	149.7	-0.1 (-0.1)	149.8	-	149.7	-0.1 (-0.1)
	7.0	141.8	141.8	-	141.8	-	141.8	-
	8.5	133.7	133.7	-	133.7	-	133.7	-
	9.0	131.2	131.2	-	131.2	-	131.2	-

Notes: See Table 7.2 for Limit State Notation
*L*transfer = 0 (section at end of debonded length)
*L*transfer = 60 ϕ (section at 60 strand diameters from debonded length toward midspan)

7.7 EFFECT OF ALLOWABLE TENSILE STRESS AT SERVICE

Prior to completion of Phase 3 of this project (Hueste et al. 2003c), this study conducted a preliminary assessment of the impact of revising critical design criteria with the objective of increasing the economy of HSC prestressed girders. As noted earlier, current specifications provide allowable stresses that were developed based on the mechanical properties of NSC of 6000 psi or less. These values that are traditionally conservative for standard designs using NSC may not be appropriate for HSC designs. Because prestressed concrete design is often governed by the allowable stresses, the effects of the allowable stresses on the required number of strands and, consequently, on the span capability were studied.

The results of the parametric study showed that the allowable tensile stress limit is critical because it controls the designs (number of strands required) for most of the cases for longer spans. Based on review of current allowable stresses (see Section 2.5) and considering the HSC Louetta Bridge design (Ralls 1995), an allowable tensile stress of $7.5\sqrt{f'_c}$ (where f'_c is in psi units) was selected for this preliminary assessment. This stress limit can also be compared to the modulus of rupture for HSC determined in Phase 1 of this study (Hueste et al. 2003b), which was found to have a best-fit equation of $10\sqrt{f'_c}$, with a lower bound value of about $8\sqrt{f'_c}$. Note that Phase 3 focused on assessing the impact on field curing conditions on the compressive strength and modulus of rupture of HSC (Hueste et al. 2002d). However, these results were not available at the time this study took place.

The parametric study showed that HSC prestressed bridge girder designs are often controlled by the compressive stress limits. In addition, Phase 1 of this study showed that for HSC produced by Texas precasters, the actual concrete compressive strength at service is typically greater than that specified, where the ratio of the actual to specified f'_c ranged from 1.01 to 1.89 (Hueste et al. 2003b). However, increases in the compressive stress limits were not selected for evaluation in this study, and the allowable compressive stress was maintained as $0.45\,f'_c$ as specified in the LRFD Specifications and in ACI 318-02. The reason for this is that the current limits for the compressive stresses were established to limit excessive creep, camber, or other local strains. The compressive stress limits for sustained loads ($0.4\,f'_c$ to $0.45\,f'_c$) are generally in the linear range of behavior for NSC. An increase in the stress limit to 0.6 f'_c is allowed for load cases including transient loads. These limits were developed for NSC, and more studies are needed to evaluate whether these limits are applicable to HSC. Assuming that the same coefficients are appropriate for the compressive stress limits for HSC prestressed members, it is not conservative to assume an overstrength will be provided in the design phase because production practices may change among precasters over time and this overstrength is not a requirement. Potentially, the actual strength gain can be utilized by tailoring designs based on strength data for a typical concrete mixture used by the selected precaster. However, the precaster may not be identified in the initial design stage and so this may not always be practical.

7.7.1 Impact on the Controlling Limit States

Tables 7.26 through 7.31 provide controlling limit states for different allowable tensile stresses at service for Type IV beams with spans from 90 ft. to maximum span lengths at 10 ft. intervals. A separate table is provided for each girder spacing considered. Different concrete classes are considered, and all cases are for 0.6 in. diameter strands with designs according to the LRFD Specifications. The controlling limit states are defined as the limit state that dictates the required number of strands or limits the maximum span.

Results showed that for shorter spans (in several cases up to 110 ft., and 120 ft., in one case), allowing a higher tensile stress has an important impact on the ultimate strength of the beams because increasing the tensile stress limit resulted in a reduction of the number of strands required for a given span length. As girder spacings decrease and concrete strengths increase, the flexural strength becomes more critical.

For a given longer span length, except for maximum span lengths, the use of f_t = $7.5\sqrt{f'_c}$ resulted in designs that were controlled by the tensile stress limit, as was the case when using $f_t = 6\sqrt{f'_c}$. However, fewer strands were required when using the higher tensile stress limit (see Sections 7.7.2 and 7.7.3).

Table 7.26. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 4.25 ft.).

f'_c (psi)	Controlling Limit State			
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	129.5	f(t) Total Load*	130	f(t) Total Load
	-	-	132.0	f(t) Total Load*
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t)_ T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	147.8	f(t) Total Load*	150	f(t) Total Load
	-	-	150.5	f(t) Total Load*
10000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	Flexural Strength / f(t) T L
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	160	f(t) Total Load	160	f(t) Total Load
	160.8	f(t) Total Load / f(c) T D L	162.8	f(t) T L / f(c) T D L
12000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	Flexural Strength / f(t) T L	100	Flexural Strength
	110	f(t) Total Load	110	Flexural Strength / f(t) T L
	120	f(t) Total Load	120	Flexural Strength / f(t) T L
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	160	f(t) Total Load	160	f(t) Total Load
	161.6	f(t) Total Load [e]	164.6	f(t) Total Load [e]

Table 7.27 Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 5 ft.).

f'_c	Controlling Limit State			
(psi)	**Length** (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	129.6	f(t) Total Load*	130	f(t) Total Load
	-	-	131.9	f(t) Total Load*
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	143.8	f(t) Total Load*	146.4	f(t) Total Load*
10000	90	Flexural Strength	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	Flexural Strength / f(t) T L
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	155.0	f(t) Total Load/f(c) T D L	157.0	f(t) T L / f(c) T D L
12000	90	Flexural Strength	90	Flexural Strength
	100	Flexural Strength / f(t) T L	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	Flexural Strength / f(t) T L
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	150	f(t) Total Load	150	f(t) Total Load
	155.4	f(t) Total Load [e]	158.3	f(t) Total Load [e]

Table 7.28. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 5.75 ft.).

f'$_c$ (psi)	**Controlling Limit State**			
	Length (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	125.0	f(t) Total Load*	127.3	f(t) Total Load*
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	138.6	f(t) Total Load*	140	f(t) Total Load
	-	-	141.1	f(t) Total Load*
10000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	Flexural Strength / f(t) T L	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	149.4	f(t) Total Load [e]	150	f(t) Total Load
	-	-	151.8	f(t) T L / f(c) T D L
12000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	Flexural Strength / f(t) T L	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	149.8	f(t) Total Load [e]	150	f(t) Total Load
	-	-	152.7	f(t) Total Load [e]

Table 7.29. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 7 ft.).

f'c	**Controlling Limit State**			
(psi)	**Length** (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	118.2	f(t) Total Load*	120	f(t) Total Load
	-	-	120.4	f(t) Total Load*
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	131.1	f(t) Total Load*	133.5	f(t) Total Load*
10000	90	Flexural Strength / f(t) T L	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	141.5	f(t) Total Load [e]	143.9	f(t) Total Load [e] & f(c) T D L
12000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	140	f(t) Total Load	140	f(t) Total Load
	141.8	f(t) Total Load [e]	144.5	f(t) Total Load [e]

Table 7.30. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 8.5 ft.).

f'_c	Controlling Limit State			
(psi)	**Length** (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	f(t) Total Load
	100	f(t) Total Load	100	f(t) Total Load
	109.4	f(t) Total Load*	110	f(t) Total Load
	-	-	113.3	f(t) Total Load*
8000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	123.4	f(t) Total Load*	125.6	f(t) Total Load *
10000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	133.0	f(t) Total Load*	135.7	f(t) Total Load [e]
12000	90	Flexural Strength / f(t) T L	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	133.7	f(t) Total Load [e]	136.2	f(t) Total Load [e]

Table 7.31. Controlling Limit States for Different Allowable Tensile Stresses at Service (Girder Spacing = 9 ft.).

f'_c	Controlling Limit State			
(psi)	**Length** (ft.)	$f_t = 6\sqrt{f'_c}$	**Length** (ft.)	$f_t = 7.5\sqrt{f'_c}$
6000	90	f(t) Total Load	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	109.2	f(t) Total Load*	110	f(t) Total Load
	-	-	111.2	f(t) Total Load*
8000	90	Flexural Strength / f(t) T L	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	121.0	f(t) Total Load*	123.3	f(t) Total Load*
10000	90	Flexural Strength / f(t) T L	90	Flexural Strength / f(t) T L
	100	f(t) Total Load	100	f(t) Total Load
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	130.1	f(t) Total Load*	132.9	f(t) T L / f(c) T D L
12000	90	Flexural Strength / f(t) T L	90	Flexural Strength
	100	f(t) Total Load	100	Flexural Strength / f(t) T L
	110	f(t) Total Load	110	f(t) Total Load
	120	f(t) Total Load	120	f(t) Total Load
	130	f(t) Total Load	130	f(t) Total Load
	131.2	f(t) Total Load [e]	133.7	f(t) Total Load [e]

7.7.2 Impact on the Number of Strands

Tables 7.32 through 7.37 show the differences for the number of strands required for spans from 90 ft. to maximum span lengths at 10 ft. intervals designed with two different allowable tensile stress limits ($f_t = 6\sqrt{f'_c}$ and $f_t = 7.5\sqrt{f'_c}$). The calculations were performed for Type IV beams with 0.6 in. diameter strands designed using the LRFD Specifications. For the same span and for girder spacings up to 7 ft., designs using $f_t = 7.5\sqrt{f'_c}$ required between two to eight fewer strands than for those designs using $f_t = 6\sqrt{f'_c}$ (approximately two for spans up to 120 ft., four for spans from 130 to 150 ft., and six to eight for spans up 160 ft.). For the same span and for girder spacings greater than 7 ft., designs using $f_t = 7.5\sqrt{f'_c}$ also required between two to eight fewer strands than for designs using $f_t = 6\sqrt{f'_c}$ (approximately two for spans up to 110 ft., four for spans of 120 ft., and six to eight for spans of 130 ft.). The percentage reduction in the number of strands when using the larger allowable tensile stress ranged from 0 to 12 percent.

Table 7.32. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 4.25 ft.).

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		**Difference**
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	**No. Strands**
6000	90	14	90	14	0
	100	18	100	18	0
	110	24	110	22	-2
	120	28	120	28	0
	129.5	34	130	34	-
	-	-	132.0	34	-
8000	90	14	90	14	0
	100	18	100	16	-2
	110	22	110	22	0
	120	28	120	26	-2
	130	34	130	32	-2
	140	42	140	40	-2
	147.8	50	150	50	-
	-	-	150.5	50	-
10000	90	14	90	14	0
	100	18	100	16	-2
	110	22	110	20	-2
	120	28	120	26	-2

Table 7.32. Continued.

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Difference
	Length (ft.)	No. Strands	Length (ft.)	No. Strands	No. Strands
	130	34	130	32	-2
	140	42	140	40	-2
	150	52	150	50	-2
	160	70	160	64	-6
	160.8	74	162.8	70	-
12000	90	14	90	14	0
	100	18	100	18	0
	110	22	110	20	-2
	120	26	120	24	-2
	130	34	130	32	-2
	140	42	140	38	-4
	150	52	150	48	-4
	160	70	160	62	-8
	161.6	76	164.6	76	-

Table 7.33. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 5 ft.).

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Difference No. Strands
	Length (ft.)	No. Strands	Length (ft.)	No. Strands	
6000	90	16	90	16	0
	100	20	100	18	-2
	110	26	110	24	-2
	120	32	120	30	-2
	129.6	38	130	38	-
	-	-	131.9	38	-
8000	90	16	90	16	0
	100	20	100	18	-2
	110	24	110	24	0
	120	32	120	32	0
	130	38	130	36	-2
	140	48	140	46	-2
	143.8	52	146.4	52	-
10000	90	16	90	16	0
	100	20	100	18	-2
	110	24	110	22	-2
	120	30	120	28	-2
	130	38	130	36	-2
	140	48	140	44	-4
	150	62	150	56	-6
	155.0	76	157.0	72	-
12000	90	16	90	16	0
	100	18	100	18	0
	110	24	110	22	-2
	120	30	120	28	-2
	130	38	130	34	-4
	140	46	140	44	-2
	150	60	150	56	-4
	155.4	76	158.3	76	-

Table 7.34. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 5.75 ft.).

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		**Difference No. Strands**
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	18	90	16	-2
	100	22	100	20	-2
	110	28	110	26	-2
	120	34	120	34	0
	125.0	38	127.3	38	-
8000	90	16	90	16	0
	100	22	100	20	-2
	110	28	110	26	-2
	120	34	120	32	-2
	130	44	130	40	-4
	138.6	52	140	52	-
	-	-	141.1	52	-
10000	90	16	90	16	0
	100	20	100	20	0
	110	26	110	24	-2
	120	34	120	32	-2
	130	42	130	40	-2
	140	54	140	50	-4
	149.4	76	150	68	-
	-	-	151.8	74	-
12000	90	16	90	16	0
	100	20	100	20	0
	110	26	110	24	-2
	120	32	120	30	-2
	130	42	130	38	-4
	140	54	140	50	-4
	149.8	76	150	66	-
	-	-	152.7	76	-

Table 7.35. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 7 ft.).

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	20	90	18	-2
	100	26	100	24	-2
	110	32	110	30	-2
	118.2	38	120	38	-
	-	-	120.4	38	-
8000	90	18	90	18	0
	100	24	100	22	-2
	110	32	110	30	-2
	120	40	120	38	-2
	130	52	130	48	-4
	131.1	52	133.5	52	0
10000	90	18	90	18	0
	100	24	100	22	-2
	110	30	110	28	-2
	120	40	120	36	-4
	130	50	130	48	-2
	140	70	140	64	-6
	141.5	76	143.9	76	-
12000	90	18	90	18	0
	100	24	100	22	-2
	110	30	110	28	-2
	120	38	120	36	-2
	130	50	130	46	-4
	140	70	140	62	-8
	141.8	76	144.5	76	-

Table 7.36. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 8.5 ft.).

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	22	90	22	0
	100	30	100	28	-2
	109.4	36	110	36	-
	-	-	113.3	38	-
8000	90	22	90	20	-2
	100	28	100	26	-2
	110	38	110	34	-4
	120	48	120	46	-2
	123.4	52	125.6	52	-
10000	90	22	90	20	-2
	100	28	100	26	-2
	110	36	110	34	-2
	120	48	120	44	-4
	130	64	130	60	-4
	133.0	74	135.7	76	-
12000	90	20	90	20	0
	100	28	100	26	-2
	110	36	110	32	-4
	120	46	120	42	-4
	130	64	130	58	-6
	133.7	76	136.2	76	-

Table 7.37. Number of Strands for Different Allowable Tensile Stresses at Service (Girder Spacing = 9 ft.).

f'_c (psi)	Number of Strands				
	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Difference No. Strands
	Length (ft.)	**No. Strands**	**Length** (ft.)	**No. Strands**	
6000	90	24	90	22	-2
	100	30	100	30	0
	109.2	38	110	38	-
			111.2	38	-
8000	90	22	90	22	0
	100	30	100	28	-2
	110	40	110	36	-4
	120	52	120	48	-4
	121.0	52	123.3	52	-
10000	90	22	90	22	0
	100	30	100	28	-2
	110	38	110	36	-2
	120	50	120	48	-2
	130	72	130	66	-6
	130.1	72	132.9	74	-
12000	90	22	90	22	0
	100	28	100	26	-2
	110	38	110	34	-4
	120	50	120	46	-4
	130	72	130	64	-8
	131.2	76	133.7	76	-

7.7.3 Impact on the Controlling Limit States for Maximum Span Lengths

Table 7.38 shows the controlling limit states for different allowable tensile stresses at service for maximum span length, and for different concrete classes and girder spacings. The calculations were performed for Type IV beams with 0.6 in. diameter strands designed using the LRFD Specifications. To further study the impact of increasing the allowable tensile stress at service, additional designs were analyzed using $f_t = 8\sqrt{f'_c}$. Maximum span lengths with their respective number of strands, initial concrete strengths, and controlling limit states designed using $f_t = 8\sqrt{f'_c}$ are shown in Tables D.25 and D.26 in Appendix D.

Table 7.38. Controlling Limit States for Maximum Span Lengths for Different Allowable Tensile Stresses at Service.

f'_c (psi)	Girder Spacing (ft.)	Controlling Limit State	
		$f_t = 6\sqrt{f'_c}$	$f_t = 7.5\sqrt{f'_c}$
6000	4.25	f(t) Total Load*	f(t) Total Load*
	5.00	f(t) Total Load*	f(t) Total Load*
	5.75	f(t) Total Load*	f(t) Total Load*
	7.00	f(t) Total Load*	f(t) Total Load*
	8.50	f(t) Total Load*	f(t) Total Load*
	9.00	f(t) Total Load*	f(t) Total Load*
8000	4.25	f(t) Total Load*	f(t) Total Load*
	5.00	f(t) Total Load*	f(t) Total Load*
	5.75	f(t) Total Load*	f(t) Total Load*
	7.00	f(t) Total Load*	f(t) Total Load*
	8.50	f(t) Total Load*	f(t) Total Load*
	9.00	f(t) Total Load*	f(t) Total Load*
10000	4.25	f(t) T L / f(c) T D L	f(t) T L / f(c) T D L
	5.00	f(t) T L / f(c) T D L	f(t) T L / f(c) T D L
	5.75	f(t) Total Load [e]	f(t) T L / f(c) T D L
	7.00	f(t) Total Load [e]	f(t) T L [e] & f(c) T D L
	8.50	f(t) Total Load*	f(t) Total Load [e]
	9.00	f(t) Total Load*	f(t) T L / f(c) T D L
12000	4.25	f(t) Total Load [e]	f(t) Total Load [e]
	5.00	f(t) Total Load [e]	f(t) Total Load [e]
	5.75	f(t) Total Load [e]	f(t) Total Load [e]
	7.00	f(t) Total Load [e]	f(t) Total Load [e]
	8.50	f(t) Total Load [e]	f(t) Total Load [e]
	9.00	f(t) Total Load [e]	f(t) Total Load [e]

Note: See Table 7.2 for Limit State Notation

Basically, three different trends were observed. For f'_c up to 8000 psi and for all girder spacings, maximum span lengths are limited by the same controlling limit states (the tensile limit at service but initially the release compressive limit) when the limit for the tensile stress at service changes from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$. A reduction in the number of strands is possible because the tensile limit was increased. However, because the maximum spans are controlled by the tensile limit, increases in maximum spans were found. The same trend was observed when the tensile stress was increased to $f_t = 8\sqrt{f'_c}$ because the tensile stress at service also controls maximum span lengths for this case (see Tables D.25 and D.26 in Appendix D).

For an f'_c of 10000 psi, two major trends were observed for the controlling limit states due to raising the tensile limit at service: (1) the controlling limit state changed from the tensile limit at service to another tensile limit at service, and (2) the controlling tensile limit state that occurs when no additional strands can be used (because it does not produce a gain in length) remained the same. For these two cases, increases in maximum span lengths were observed.

When $8\sqrt{f'_c}$ was used, the first trend explained above changed. In this case, the controlling limit state changed from the tensile limit at service to the compressive limit at the intermediate stage. Maximum spans still increased. However, increases in the tensile stress beyond $8\sqrt{f'_c}$, where the controlling limit state is the compressive limit, resulted not only in a reduction of the number of strands but also in span length. For the second trend, the controlling limit states remained the same for most of the cases when the tensile limit increased. However, a new trend for only one case was observed when the tensile stress was raised from $7.5\sqrt{f'_c}$ to $8\sqrt{f'_c}$. The controlling limit state changed from the compressive limit to one in which the release compressive stress primarily controls the number of strands, followed by the tensile limit at service.

For an f'_c of 12000 psi, the tensile limit at service that occurs because no additional strands can be used controlled maximum span lengths designed by $f_t = 6\sqrt{f'_c}$, $f_t = 7.5\sqrt{f'_c}$, and $f_t = 8\sqrt{f'_c}$ (see Tables 7.38, D.25, and D.26).

7.7.4 Impact on Maximum Span Lengths

Table 7.39 shows maximum span lengths for different allowable tensile stresses at service and for different concrete classes and girder spacings. The calculations were performed for Type IV beams with 0.6 in. diameter strands designed using the LRFD Specifications. To further study designs that were controlled by the tensile limit at service, additional designs were analyzed using $f_t = 8\sqrt{f'_c}$. Maximum span lengths with their respective number of strands, initial concrete strengths, and controlling limit states for designs using $f_t = 8\sqrt{f'_c}$ are shown in Tables D.25 and D.26 in Appendix D.

Table 7.39. Maximum Span Lengths for Different Allowable Tensile Stresses at Service.

f'_c (psi)	Girder Spacing (ft.)	$f_t = 6\sqrt{f'_c}$		$f_t = 7.5\sqrt{f'_c}$		Change in Max. Span ft. (%)
		Max. Span (ft.)	No. Strands	Max. Span (ft.)	No. Strands	
6000	4.25	129.5	34	132.0	34	2.5 (1.9)
	5.00	129.6	38	131.9	38	2.3 (1.8)
	5.75	125.0	38	127.3	38	2.3 (1.8)
	7.00	118.2	38	120.4	38	2.2 1.8)
	8.50	111.3	38	113.3	38	2.0 (1.8)
	9.00	109.2	38	111.2	38	2.0 (1.9)
8000	4.25	147.8	50	150.5	50	2.7 (1.8)
	5.00	143.8	52	146.4	52	2.6 (1.8)
	5.75	138.6	52	141.1	52	2.5 (1.8)
	7.00	131.1	52	133.5	52	2.4 (1.8)
	8.50	123.4	52	125.6	52	2.3 (1.8)
	9.00	121.0	52	123.3	52	2.2 (1.8)
10000	4.25	160.8	74	162.8	70	2.0 (1.2)
	5.00	155.0	76	157.0	72	2.1 (1.3)
	5.75	149.4	76	151.8	74	2.3 (1.6)
	7.00	141.4	76	143.9	76	2.5 (1.7)
	8.50	133.0	74	135.7	76	2.7 (2.0)
	9.00	130.1	72	132.9	74	2.8 (2.1)
12000	4.25	161.6	76	164.6	76	3.0 (1.9)
	5.00	155.4	76	158.3	76	3.0 (1.9)
	5.75	149.8	76	152.7	76	2.9 (1.9)
	7.00	141.8	76	144.5	76	2.7 (1.9)
	8.50	133.7	76	136.2	76	2.6 (1.9)
	9.00	131.2	76	133.7	76	2.5 (1.9)

Basically, three different trends were observed. For concrete strengths up to 8000 psi and for all girder spacings, where maximum span lengths are controlled by the same controlling limit states (release compressive limit followed by the tensile limit at service), the increase in the allowable tensile stress at service from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ resulted in increases in maximum spans up to 2.7 ft. (1.8 percent). The same trend was observed for designs using $f_t = 8\sqrt{f'_c}$, but in this case, the increases in maximum span lengths go up to 2.5 percent (3.3 ft.). In all these cases that are compared, the same number of strands were used because the release compressive limit controlled the number of strands.

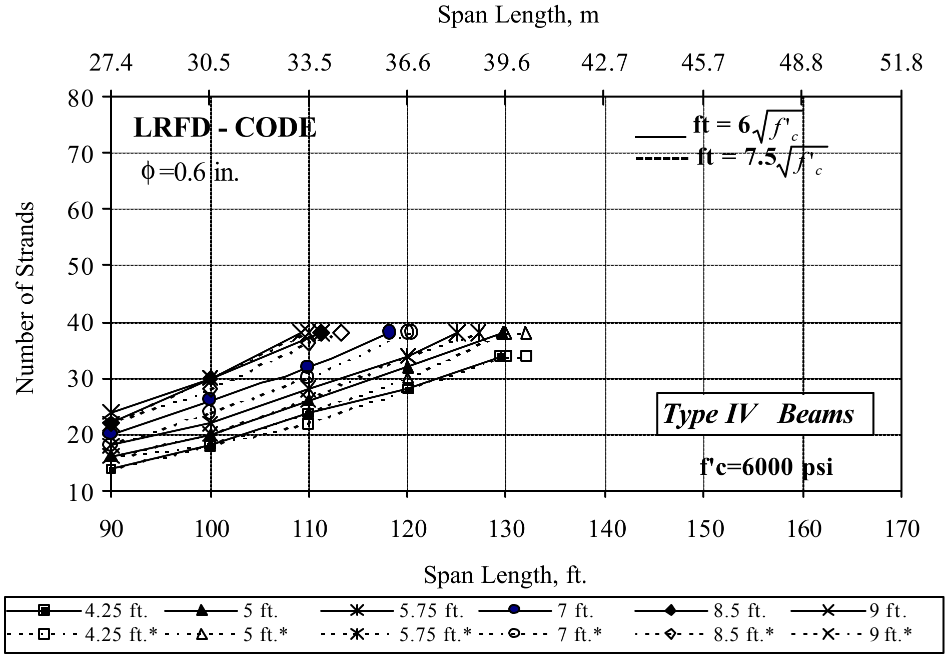
For a concrete strength of 10000 psi, increases in maximum span length were determined when raising the tensile limit at service. When the controlling limit state changed from the tensile limit at service to the compressive limit at the intermediate stage, increases of up to 2.8 ft. (2.1 percent) in span were observed. When the controlling limit state is tension at service where no additional strands can be used, the increase in the tensile stress limit led to increases in span up to 2.7 ft. (2.0 percent).

When the tensile stress was increased to $8\sqrt{f'_c}$, the first trend explained above was different. In this case, the controlling limit state changed to the compressive limit at the intermediate stage with no available tensile stress and maximum spans increased up to 3.3 ft. (2.3 percent). However, the resulting controlling limit state (compressive limit at the intermediate stage) indicates that an increase in the tensile stress (more than $8\sqrt{f'_c}$) would result not only in a reduction of the number of strands but also in length, as occurred in designs for U54 Beams using concrete strengths up to 10000 psi with girder spacings less than 11.5 ft. For the second trend, increases up to 3.5 ft. (2.6 percent) were found (see Tables D.25 and D.26). For the new trend, where basically the compressive limit at the intermediate stage changed to the release compressive limit, an increase in maximum span length of 3.9 ft. (3 percent) was found.

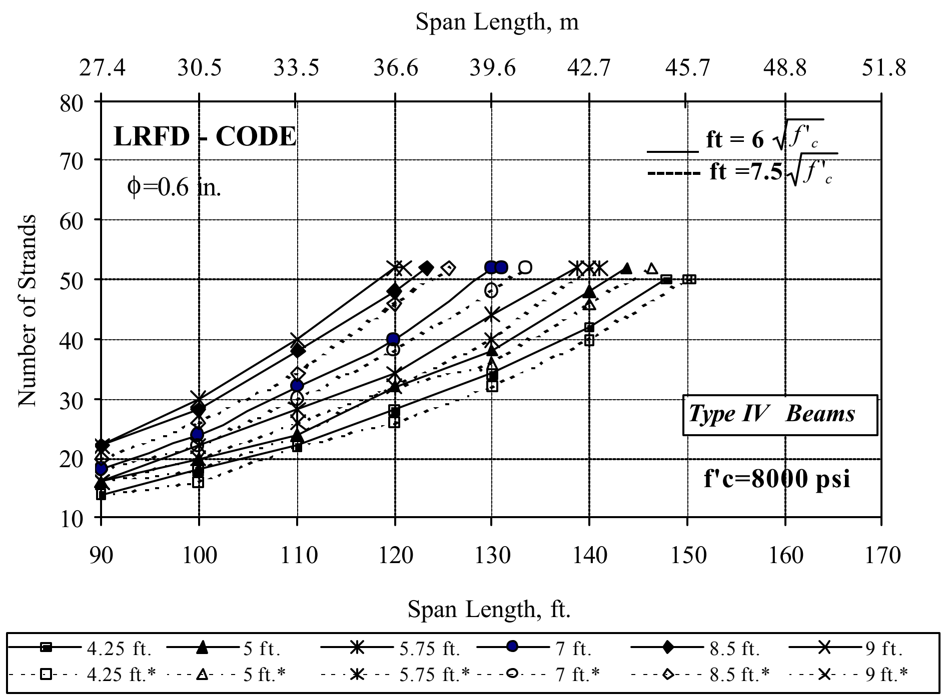
For a concrete strength of 12000 psi where the tensile limit at service (that occurs because no additional strands can be used) controlled the maximum span, increases in maximum span lengths up to 3 ft. (1.9 percent) were determined when the tensile limit was increased from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$. When the tensile stress was increased from $6\sqrt{f'_c}$ to $8\sqrt{f'_c}$, an increases in maximum span lengths up to 2.6 percent (4 ft.) was determined (see Tables D.25, and D.26).

7.7.5 Span Capability

Figure 7.5 shows the impact on span capability of the Type IV beam designed using the AASHTO LRFD Specifications for two different allowable tensile stresses, $6\sqrt{f'_c}$ and $7.5\sqrt{f'_c}$. These figures show the trends for number of strands versus span lengths for different girder spacings and concrete strengths. There are two ways to interpret these results. On the vertical axis, each interval of allowable tensile stress represents saving between approximately two and four strands for the same span and girder spacing. The required concrete strength at release was almost constant; however, for 12000 psi of concrete strength, reductions were found (see Tables D.25 and D.26 provided in Appendix D). On the horizontal axis, each interval of allowable tensile stress represents an increase in span capability of approximately 2 to 4 ft. for the same number of strands and girder spacings.

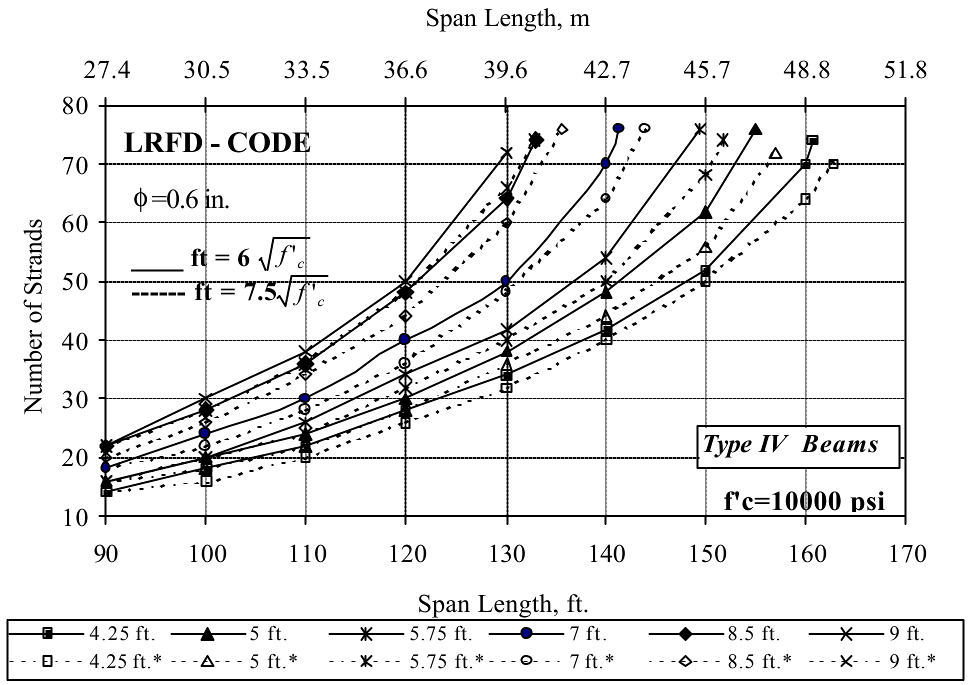


(a) f'_c = 6000 psi

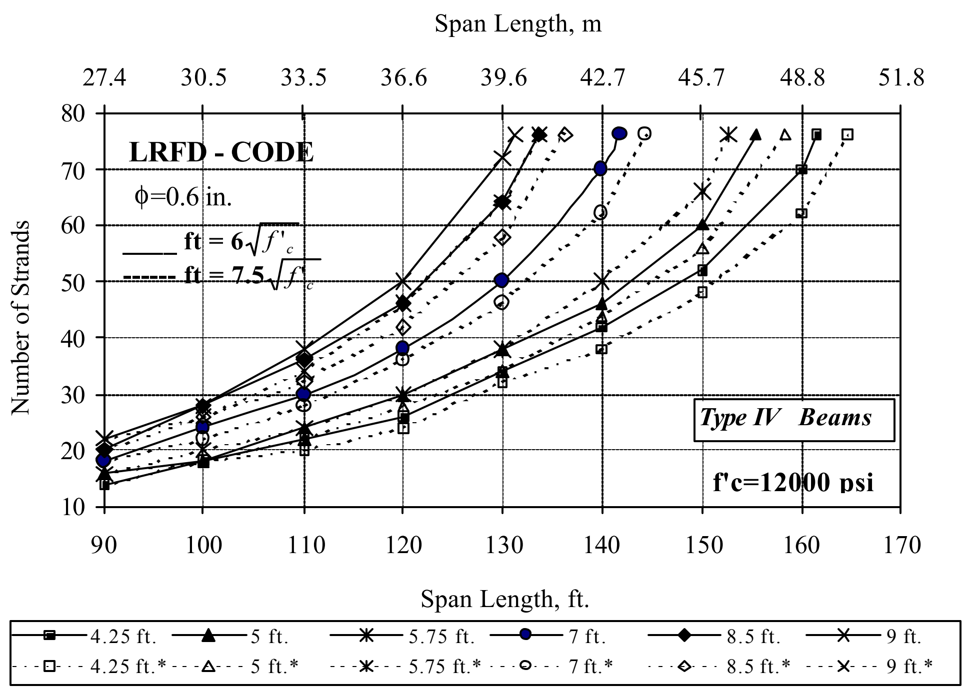


(b) f'_c = 8000 psi

Figure 7.5. Number of Strands versus Span Lengths for Different Allowable Tensile Stresses (LRFD Specifications, Strand Diameter = 0.6 in.).



(c) f'_c = 10000 psi



(d) f'_c = 12000 psi

Figure 7.5. Continued.

8 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 SUMMARY

This report summarizes Phase 2 of TxDOT Research Project 0-2101, "Allowable Stresses and Resistance Factors for High Strength Concrete." The objective of this project was to evaluate the allowable stresses and resistance factors in the AASHTO LRFD Specifications for design of HSC girders used in Texas bridges. Hueste et al. (2003a) summarized the complete project. Phase 1 of this project (Hueste et al. 2003b) evaluated the applicability of current prediction equations for estimation of mechanical properties of HSC and also determined statistical parameters for mechanical properties of HSC. The HSC samples for Phase 1 were collected from three Texas precasters that manufacture HSC prestressed bridge girders. Phase 3 of this project assessed the impact of different curing conditions on the compressive and flexural strength of HSC mixtures used for prestressed girders in Texas (Hueste et al. 2003c). The portion of the research project addressed by this study (Phase 2) includes defining the current state of practice for design of HSC prestressed girders and identifying critical design parameters that limit the design of typical HSC prestressed bridge girders. There are three specific research objectives for this study: (1) to determine the current state of practice for HSC prestressed bridge girders across the United States, (2) to evaluate the controlling limit states for the design of HSC prestressed bridge girders and identify areas where some economy in design may be gained, and (3) to conduct a preliminary assessment of the impact of revising critical design criteria with an objective of increasing the economy of HSC prestressed girders.

The first objective was accomplished through a literature search and survey. The literature search included review of design criteria for both the AASHTO Standard and LRFD Specifications and relevant case studies of the performance of HSC prestressed bridge girders. In addition, a survey was conducted to gather information and document critical aspect of current design practices for HSC prestressed bridges. Researchers collected responses from 41 state DOTs and two private organizations, giving a 74 percent response rate to the survey.

The second objective was accomplished by conducting a parametric study for single-span HSC prestressed bridge girders to primarily investigate the controlling flexural limit states for both the AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b). The selected variables for the overall parametric study are shown in Table 8.1. AASHTO Type IV and Texas U54 Beams were considered. The effects of changes in concrete strength, strand diameter, girder spacing and span length were evaluated. Several case study bridges with U54 and Type IV beams were designed using the Standard Specifications, and the results were compared with those from PSTRS14 (TxDOT 1980) to ensure consistency of the parametric study with TxDOT design practices. Based on the results from the parametric study, the limiting design criteria for HSC prestressed U54 and Type IV girders using both the AASHTO Standard and LRFD Specifications for Highway Bridges were identified.

Table 8.1. Design Parameters.

Variable	Description / Selected Values
Codes	AASHTO Standard and LRFD Specifications (AASHTO 2002 a,b)
Concrete Strength (psi)	6000, 8000, 10000, and 12000 (f'_{ci} is initially set at 0.75 f'_c, but allowed to vary up to f'_c)
Girder Sections	Texas U54 and AASHTO Type IV
Girder Spacing for U54 Beams (ft)	8.5, 10, 11.5, 14.0, and 16.67
Girder Spacing for Type IV Beams (ft)	4.25, 5, 5.75, 7.0, 8.5. and 9.
Spans (L)	90 ft. to maximum span at 10 ft. intervals
Diameter Strands (inches)	0.5 and 0.6

The third research objective was accomplished by making a preliminary evaluation of the impact of raising the allowable tensile stress for service conditions. At the time of this study, the results of the experimental work related to curing effects on concrete strength were not yet available (see Hueste et al. 2003c). Therefore, the tensile stress limit selected for this evaluation was based on the current limit for uncracked sections provided by the ACI 318 code (ACI Comm. 318 2002) and the limit used for a specific case study bridge (Ralls 1995). The potential increase in bridge span lengths based on the increased tensile stress limit at service was evaluated. In addition, other refinements in the flexural design to achieve increased span lengths or to reduce the number of strands required for a specific span were evaluated.

8.2 CONCLUSIONS

Major conclusions derived from this study are noted below. The conclusions are organized according to the major research objectives.

8.2.1 Current State of Practice

Codes and Documents

For the 41 DOTs involved in the survey, 78 percent are currently using the AASHTO Standard Specifications, 44 percent are using the AASHTO LRFD Specifications, and 22 percent are using both specifications. Most states that use the LRFD Specifications only partially implement the specification; most states plan complete implementation in the period of 2003 to 2007. The survey shows that about one-third of the state DOTs use additional documents and references for the design of prestressed concrete bridge girders and HSC members. These documents and references include the PCI Bridge Design Manual (1997), some publications on HSC issued by the Portland Cement Association, bridge design manuals developed by individual states, software programs developed by some states or software companies, and other reports and texts.

Prevalence of HSC Prestressed Bridge Girders

Of the 41 responding DOTs, 68 percent use HSC prestressed girders for 0 to 50 percent of their total construction, 15 percent of the responding DOTs use HSC prestressed girders for 51 to 80 percent of their total construction, and 17 percent of the responding DOTs use HSC prestressed girders for 81 to 100 percent of their total construction.

Specified Concrete Strengths

The most typical range for the specified concrete strength at transfer (f'_{ci}) is 4000 to 7000 psi, while the specified concrete strength at service (f'_c) typically ranges from 5000 to 8500 psi. Only 15 percent of the DOTs utilize a higher concrete strength at service (10000 psi) for some cases, and just 2 percent of the DOTs utilize a concrete strength at service of 12000 psi.

Impact of Required Transfer Strength

The respondents indicated that the specified release strength tends to be critical for prestressed concrete girder production because mixture designs are governed by the required concrete strength at release. Twenty-two of the state DOTs have observed that high initial strength requirements have led to an overstrength in actual concrete strength at service. In this study, the definition of HSC is concrete with specified compressive strengths for design of 6000 psi or greater, made without using exotic materials or techniques (ACI 363 1997). However, most of the responses indicate that high transfer strengths require special materials or techniques like accelerated curing. Two methods are mentioned to obtain HSC. First is to obtain a high initial concrete strength (within 18 hours to 2 days) using high early cement and/or heat curing. In this case, the final strengths tend to level off quickly (around 7 days) and the strength gain is not significant. Second is the common method of curing at ambient conditions, which tends to provide final strengths higher than those specified in designs. In this case, if precasters focus on achieving the high initial concrete strength demands with ambient curing methods, then the specified 28-day strength is met quickly and larger concrete strengths are typically achieved at 28 days.

Concerns Related to the Use of HSC

The survey indicates that almost one-half of the DOTs have some concerns related to the use of HSC. Some of the concerns related to this study follow:

- Girder transportation limits maximum span lengths. However, design recommendations mentioned that a recent project in San Angelo, Texas, utilized HSC with a concrete strength (f'_c) of 14000 psi to construct a 153 ft. span with Type IV beams (TxDOT 2001a). Moreover, the same document states that beams up to 150 ft. have been successfully transported, although at a premium cost.
- Several DOTs are reluctant to use f'_c values higher than 8500 psi. There is a concern that design parameters in the AASHTO specifications need to be modified for use with HSC because the current design equations in the AASHTO specifications for prestressed concrete members are based on mechanical properties of normal concrete strengths of 6000 psi or less.

- Initial cracking of girders during pouring and before release is a concern. However, TxDOT practice indicates that cracking at release is not a major problem in practice because transfer is a temporary condition. If a crack occurs in the top of the beam at the end regions, it will close when the concrete slab is poured.

Suppliers

The survey shows that a single DOT may be served by one to seven precasters that can supply HSC girders. HSC precasters not only supply to their own state DOT but may also supply to other state DOTs. In addition, in some areas, there is an unavailability of suitable aggregates and/or no qualified precasters to produce HSC prestressed girders.

Adjustments to Design Specifications for HSC Prestressed Bridge Girders

The survey indicates that most DOTs have not made adjustments to the design specifications for the design of HSC prestressed bridge girders. Of the seven DOTs that have made modifications, in-house design documents include modifications for the equation for the modulus of elasticity, the allowable stresses, and the equation for losses, creep, and camber.

Typical Bridges with HSC Prestressed Bridge Members

The HSC prestressed girder types that are most popular among the responding DOTs include the AASHTO beam (26 states), followed by the bulb beam (23 states), and the box girder (11 states). Voided slabs (6 states), slabs (4 states), double T-beams (4 states), and closed box CIP beams (1 state) are the structural types with less use, although the closed box CIP girder is used for long spans (typically up to 150 ft.). The Texas U-beams are used not only in Texas, but also in Colorado and New Mexico.

Slab, voided slab, and double T-beams are more prevalent for shorter span lengths. The typical range for shorter span lengths is from approximately 30 to 60 ft., and the typical range for specified concrete strengths at service (f'_c) varies from approximately 3500 to 6000 psi. Note that this strength range would not be considered to be HSC for this study.

Closed box CIP beams, AASHTO beams, bulb beams, and box beams are more prevalent for longer span lengths. The typical range for longer span lengths is from approximately 60 to 150 ft., and the typical range for specified concrete strengths at service (f'_c) varies from approximately 6000 to 10000 psi. More details are provided in Section 4.3.

8.2.2 Parametric Study Using Current Specifications

Researchers derived the following conclusions for the parametric study. This study focused only on limit states related to flexure for service and ultimate conditions. Additional design limit states were not evaluated. TxDOT currently uses an HS25 truck loading for a number of designs. The loading used in this study was based on the specified loads in the AASHTO specifications, which reference an HS20 truck loading.

8.2.2.1 Trends for U54 Girders

Controlling Limit States

1. The ultimate flexural limit state does not produce an impact in terms of limiting maximum span lengths because this limit state only controls shorter span lengths (90 ft. and, in some cases, 100 ft.).
2. The concrete tensile stress at full service loads controlled the number of strands required for longer spans, except for the maximum span lengths, whether they are designed under the Standard or the LRFD Specifications.
3. The concrete compressive stress due to total dead loads (sustained loads) controlled the number of strands required for maximum span lengths of beams designed under both the Standard and LRFD Specifications. The exception is when the allowable tensile stress limit under total loads would be exceeded because no additional prestressing strands can be accommodated in the U54 beam section or because the stresses at the beam ends during transfer initially limit the number of strands.
4. Stresses at the beam ends at release do not control any maximum span length of U54 Beams designed under the Standard Specifications. Stresses at the beam ends at release control some maximum span lengths for U54 Beams with wider girder spacings (14 and

16.6 ft.) designed under the LRFD Specifications. These stresses become critical when 0.6 in. diameter strands and concrete strengths up to 10000 psi are used, and this effect is reduced when concrete strengths are higher (12000 psi). Consequently, release concrete stresses at the beam ends become critical for wider girder spacings using 0.6 in. diameter strands designed under the LRFD Specifications. In these cases, the U54 beam section is not fully utilized because the release stress limit significantly reduces the number of strands that can be used, and consequently, shorter maximum span lengths are obtained.

Effective Concrete Strength

The effective concrete strength is defined as the maximum f'_c above which no significant increases in maximum span length were found in the parametric study. Maximum span lengths depend not only on the concrete strength but also on the strand diameter. U54 Beams using 0.5 in. diameter strands can effectively use concrete compressive strengths up to 10000 psi when they are designed using both the Standard and LRFD Specifications. Exceptions are for beams with wider girder spacings (more than 11.5 ft.) that can fully use strengths only up to 8000 psi when they are designed using the Standard Specifications. However, designs using 0.6 in. diameter strands give the larger prestressing forces needed to fully utilize the cross section with concrete compressive strengths up to 12000 psi for both the Standard and the LRFD Specifications. These trends are summarized in Table 8.2.

Table 8.2. Effective Concrete Strength (U54 Girders).

Strand Diameter (in.)	**Girder Spacing** (ft.)	**Effective Concrete Strength at Maximum Span Length** (psi)	
		Standard	**LRFD**
0.5	$S \leq 11.5$	10000	10000
	$S > 11.5$	10000	8000
0.6	$S \leq 11.5$	12000	12000
	$S > 11.5$	12000	12000

Impact of Concrete Strength and Strand Diameter on Maximum Span Lengths

Increases in maximum spans due to raising concrete compressive strengths from 6000 to 12000 psi vary with girder spacing, strand diameter, and design specifications used, as shown in Table 8.3. Comparisons are made only for those strengths that are considered to be effective, as noted in the table. Average increases in maximum span lengths are smaller for wider girder spacings (14 and 16.6 ft.) when LRFD Specifications are used and overall when 0.5 in. diameter strands are used.

Table 8.3. Impact of Increasing Concrete Compressive Strengths (U54 Girders).

Strand Diameter (in.)	**Girder Spacing** (ft.)	**Average Increase in Max. Span Length ft. (%)**		**Effective Range of Concrete Strength** (psi)	
		Standard	**LRFD**	**Standard**	**LRFD**
0.5	$S \leq 11.5$	25 (23)	23 (20)	6000 – 10000	6000 – 10000
	$S > 11.5$	17 (17)	10 (9)	6000 – 10000	6000 – 8000
0.6	$S \leq 11.5$	40 (36)	39 (33)	6000 – 12000	6000 – 12000
	$S > 11.5$	31 (30)	24 (24)	6000 – 12000	6000 – 12000

Longer span lengths can be achieved using HSC and 0.6 in. diameter strands. However, in some cases, the additional span length requires a large amount of additional final prestress. The additional final prestressing force to be stored requires a correspondingly higher initial prestressing force, which increases the initial concrete strength requirements at transfer. In some cases, limitations on the initial concrete strength can then dictate the maximum achievable spans. Average increases in maximum spans of 14 to 15 ft. were found when U54 Beams are designed using 0.6 in. diameter strands rather than 0.5 in. diameter strands.

Impact of Specifications on Maximum Span Lengths

Maximum differences in maximum span lengths for LRFD designs relative to Standard designs are shown in Table 8.4. Comparisons are made only for those strengths that are considered to be effective when using both specifications. The trends vary with concrete strength, strand diameter, and especially with girder spacing. In general, for U54 Beams using 0.6 in. diameter strands, LRFD designs with girder spacings less than 11.5 ft. resulted in up to 7.4 ft. (5.3 percent) longer span length compared to Standard designs. However, LRFD designs with girder spacings greater than 11.5 ft. resulted in up to 13.7 ft. (11 percent) shorter span length compared with Standard designs. Longer spans are explained because the compressive stress limit due to sustained loads used in LRFD designs was increased from 0.4 f'_c to 0.45 f'_c. Shorter spans occur because the release stresses that control LRFD designs with wider girder spacings produced significant reductions in the number of strands. The same trends were found when 0.5 in. diameter strands were used; however, the differences are smaller (see Table 8.3).

Table 8.4. Maximum Differences in Maximum Span Length for LRFD Relative to Standard Specifications (U54 Girders).

Girder Spacing (ft.)	Strand Diameter = 0.5 in.				Strand Diameter = 0.6 in.			
	6000 psi	8000 psi	10000 psi	12000 psi	6000 psi	8000 psi	10000 psi	12000 psi
≤ 11.5	5.6 ft. (5.1%)	6.5 ft. (5.2%)	3.8 ft. (2.7%)	---	5.9 ft. (5.3%)	6.5 ft. (5.1%)	7.4 ft. (5.3%)	5.8 ft. (3.7%)
>11.5	4.0 ft. (4.0%)	-2.6 ft. (-2.4%)	---	---	-3.6 ft. (-3.6%)	-10.8 ft. (-9.6%)	-13.7 ft. (-11%)	-7.4 ft. (-5.4%)

Impact of Specifications on Required Number of Strands

For U54 Beams using 0.6 in. diameter strands, LRFD designs required between one and five fewer strands than for the designs using the Standard Specifications with girder spacings less than or equal to 11.5 ft. LRFD designs required between 1 and 18 more strands than designs using the Standard Specifications for wider girder spacings (14 and 16.6 ft.). The effect of the 0.8 factor included in LRFD Service III limit state (which is applied on the live load when tension under live load is being investigated) compared with the 1.0 factor considered in the Standard Specifications resulted in a reduction of strands required for the same load requirements. However, more strands were needed for designs using girder spacings greater than 11.5 ft. This larger number of strands can be explained by the larger LRFD live load demands.

Impact of Tensile Stress Limits at Release and Transfer Lengths

1. The release tensile stress at the beam ends is critical when the lower tensile stress limit at transfer of $3\sqrt{f'_{ci}}$ is used rather than the limit of $7.5\sqrt{f'_{ci}}$ (for the Standard designs) or $6.96\sqrt{f'_{ci}}$ (for the LRFD designs) used in this study. The number of strands is limited by this lower tensile limit, and consequently, maximum span lengths are dramatically reduced. Decreases up to 19.7 ft. (14 percent) in maximum span length for girder spacings less than or equal to 11.5 ft. and up to 38.6 ft. (31 percent) for girder spacings greater than 11.5 ft. were determined for designs using both the AASHTO Standard and LRFD Specifications and 0.6 in. diameter strands.
2. Evaluating stresses at the transfer length of 60 strand diameters from the end of debonding rather than at the end of the beam does not impact the maximum span length

for designs using the Standard Specifications and the upper tensile stress limit ($7.5\sqrt{f'_{ci}}$). The transfer length of 60 strand diameters resulted in increases in maximum span lengths up to 9.5 ft. (8.6 percent) only for the widest girder spacing (16.6 ft.) for designs using the LRFD Specifications and the higher tensile stress limit ($6.96\sqrt{f'_{ci}}$).

3. Considering a transfer length of 60 strand diameters resulted in increases in maximum span lengths up to 19.7 ft. (16 percent) for designs using the Standard Specifications and the lower tensile stress limit ($3\sqrt{f'_{ci}}$). A transfer length of 60 strand diameters resulted in increases in maximum span lengths up to 15.6 ft. (24 percent) for designs using the LRFD Specifications and the lower tensile stress limit ($3\sqrt{f'_{ci}}$). Theses values are representative for wider girder spacings.
4. The ultimate flexural strength was critical for maximum span lengths with the widest girder spacing (16.6 ft.) when the lower tensile stress limit at transfer of $3\sqrt{f'_{ci}}$ (with zero transfer length) was used rather than the $7.5\sqrt{f'_{ci}}$ (for the Standard designs) or $6.96\sqrt{f'_{ci}}$ (for the LRFD designs). For these cases, decreases in maximum span lengths up to approximately 37 ft. (30 percent) were determined when the limit for the tensile stress changed from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with zero transfer length) for both the LRFD and the Standard Specifications. However, these decreases in maximum span lengths were reduced up to approximately 27 ft. (22 percent) when the limit for the tensile stress changed from $7.5\sqrt{f'_{ci}}$ (with zero transfer length) to $3\sqrt{f'_{ci}}$ (with 60 transfer length) for designs using both the LRFD or Standard Specifications.

8.2.2.2 Trends for Type IV Girders

Controlling Limit States

1. The ultimate flexural limit state does not produce an impact in terms of limiting maximum span lengths because this limit state only controls shorter span lengths (90 ft. and, in some cases, 100 ft.) whether designs are under the Standard or LRFD Specifications.

2. The concrete tensile stress at service loads (full working loads) controlled the number of strands required for longer spans, except for the maximum span lengths, whether they are designed under the Standard or the LRFD Specifications.
3. The concrete compressive stress due to total dead loads (sustained loads) controlled the number of strands required for maximum span lengths of beams designed under both the Standard and LRFD Specifications. The exception is when the allowable tensile limit stress under total loads would be exceeded because no additional prestressing strands can be used with the Type IV beam section or because the stresses at the beam ends during transfer initially limit the number of strands.
4. Unlike U54 beam designs, the compressive stress at the beam ends during transfer control some of the maximum span lengths of Type IV beams with wider girder spacing designed using the Standard Specifications. Like U54 beam designs, several maximum spans are controlled by the compressive stresses at transfer when they are designed under the LRFD Specifications. However in this case, maximum span lengths for all girder spacings are limited by the compressive stress at transfer under the LRFD Specifications. These stresses become critical when 0.6 in. diameter strands and concrete strengths up to 8000 psi are used, and this effect is reduced when concrete strengths are larger (12000 psi).

Effective Concrete Strength

The effective concrete strength is defined as the maximum f'_c above which no significant increases in maximum span length were found in the parametric study. Maximum span lengths depend not only on the concrete strength but also on the strand diameter. The span capacity of Type IV beams using 0.5 in. diameter strands is promptly reached, and consequently they can effectively use concrete compressive strengths up to 10000 psi when they are designed using the Standard Specifications but only up to 8000 psi when they are designed using the LRFD Specifications. However, designs using 0.6 in. diameter strands give larger prestressing forces needed to fully utilize the concrete compressive strength up to 12000 psi when they are designed under the Standard Specifications and up to 10000 psi when they are designed under the LRFD Specifications (see Table 8.5).

Table 8.5. Effect of Strand Diameter and Strength on Maximum Span Lengths (Type IV Girders).

Strand Diameter (in.)	**Girder Spacing** (ft.)	**Effective Concrete Strength at Maximum Span Length** (psi)	
		Standard	**LRFD**
0.5	All	10000	8000
0.6	All	12000	10000

Impact of Concrete Strength and Strand Diameter on Maximum Span Lengths

Increases in maximum spans due to raising concrete compressive strengths from 6000 to 12000 psi vary with stand diameter and design specifications, as shown in Table 8.6. Comparisons are made only for those strengths that are considered to be effective as noted in the table. Average increases in maximum span lengths are smaller when LRFD Specifications are used and smaller for both specifications when 0.5 in. diameter strands are used.

Longer span lengths can be achieved using HSC and 0.6 in. diameter strands. However, in some cases, the additional span length requires a large amount of additional final prestress. The additional final prestressing force to be stored requires a correspondingly higher initial prestressing force, which increases the initial concrete strength requirements at transfer. In some cases, limitations on the initial concrete strength can then dictate the maximum achievable spans. Average increases in maximum spans of 10 and 13 ft. were found when Type IV beams were designed using 0.6 in. diameter strands rather than 0.5 in. diameter strands.

Table 8.6. Impact of Increasing Concrete Compressive Strengths (Type IV Girders).

Strand Diameter (in.)	**Girder Spacing** (ft.)	**Average Increase in Max. Span Length ft. (%)**		**Effective Range of Concrete Strength** (psi)	
		Standard	**LRFD**	**Standard**	**LRFD**
0.5	All	19 (16)	10 (8.5)	6000 – 10000	6000 – 8000
0.6	All	32 (27)	20 (20)	6000 – 12000	6000 – 10000

Impact of Specifications on Maximum Span Lengths

Maximum differences in maximum span lengths for LRFD designs relative to Standard designs are shown in Table 8.7. Comparisons are made only for those strengths that are considered to be effective for Type IV beams using both specifications. The trends vary with concrete strength, strand diameter, and especially with girder spacing. In general, for both diameter strands considered, LRFD designs with girder spacings less than 5.75 ft. resulted in up to 6.4 ft. (5 percent) longer span length compared with Standard designs. However, LRFD designs with girder spacings greater than 5.75 ft. resulted in up to 3.6 ft. (2.8 percent) decrease in span length compared with Standard designs. Longer spans are explained because the compressive stress limit due to sustained loads used in LRFD designs was increased from $0.4 f'_c$ to $0.45 f'_c$. Shorter spans occur because the release stresses that control LRFD designs resulted in significant reductions in the number of strands.

Table 8.7. Maximum Differences in Maximum Span Length for LRFD Relative to Standard Specifications (Type IV Girders).

Girder Spacing (ft.)	Strand Diameter = 0.5 in.				Strand Diameter = 0.6 in.			
	6000 psi	8000 psi	10000 psi	12000 psi	6000 psi	8000 psi	10000 psi	12000 psi
≤ 5.75	6.4 ft. (5.0%)	2.3 ft. (1.6%)	---	---	3.7 ft. (2.9%)	2.3 ft. (1.6%)	1.6 ft. (1.0%)	---
> 5.75	-0.2 ft. (-0.2%)	0.9 ft. (-0.8%)	---	---	-0.6 ft. (-0.5%)	-3.6 ft. (-2.8%)	-3.0 ft. (-2.2%)	---

Impact of Specifications on Required Number of Strands

For Type IV beams using 0.6 in. diameter strands, LRFD designs required between 0 and 18 strands more than designs using the Standard Specifications. However, more strands are needed for all girder spacings considered. This larger number of strands can be explained by the larger LRFD live load demands.

Impact of Tensile Stress Limits at Release and Transfer Lengths

1. In general, the use of the lower tensile stress limit at transfer of $3\sqrt{f'_{ci}}$ [rather than the limit of $7.5\sqrt{f'_{ci}}$ (for the Standard designs) or $6.96\sqrt{f'_{ci}}$ (for the LRFD designs) used in this study] has no impact on designs for the Type IV beam designed by either the Standard or LRFD Specifications. However, there is one case under the Standard Specifications where the use of the lower release tensile limit resulted in a 2.4 ft. (2.2 percent) reduction of the maximum span length.
2. When the transfer length of 60 strand diameters was considered in the stress checks, the controlling stress at release changed from the tensile stress to the compressive stress. This change made it possible to add two more strands and consequently gave an increase

in the span length of 2.3 ft. (2.1 percent). In this case using the lower tensile stress and the transfer length of 60 strand diameters gave the same design as if the upper release tensile limit with zero transfer length was used (see Table 7.24).

3. In general, the use of the transfer length of 60 strand diameters for the stress calculations has an impact on the maximum span lengths, especially for lower strengths. For designs with an f'_c of 6000 psi, using either the upper or lower tensile limit at release under the Standard Specifications, an increase up to 2.3 ft. (about 2 percent) in the maximum span length was found for wider girder spacings (8.5 and 9 ft.). For designs with an f'_c of 8000 psi, using the upper tensile limit and the LRFD Specifications, an increase up to 7.1 ft. (5.5 percent) in the maximum span length was found. For designs with an f'_c of 8000 psi, using the lower tensile limit at release and the LRFD Specifications, an increase up to 3.3 ft. (2.2 percent) in the maximum span length was found.

8.2.3 Impact of Raising the Allowable Tensile Stress

The third research objective was accomplished by conducting a preliminary evaluation of the impact of raising the allowable tensile stress for service conditions. At the time of this study, the results of the experimental work related to curing effects on concrete strength were not yet available (see Hueste et al. 2003c). Therefore, the tensile stress limit selected for this evaluation was based on the current limit for uncracked sections provided by the ACI 318 code (ACI Comm. 318 2002) and the limit used for a specific case study bridge (Ralls 1995). The potential increase in bridge span lengths based on the increased tensile stress limit at service was evaluated. In addition, other refinements in the flexural design to achieve increased span lengths or to reduce the number of strands required for a specific span were evaluated. This evaluation was limited to designs using the LRFD Specifications and 0.6 in. diameter strands.

8.2.3.1 Trends for U54 Girders

1. Results showed that for shorter spans (in several cases up to 100 ft., and 110 ft. in one case), an increase in the tensile stress from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ tends to lower the ultimate strength of the beams because this increase resulted in a reduction of the required number of strands for a given span length.

2. For the same span length, an increase in the tensile stress from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ resulted in between one and seven (11 for one case) fewer strands. Reductions in the required concrete strength at release were also found.
3. An increase in the tensile stress limit at service has an impact on maximum span lengths only for U54 Beams with wider girder spacings (14 and 16.6 ft.) where the tensile stress limit controls. An increase in the tensile limit at service from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ resulted in an increase in maximum span lengths up to 5.5 percent in these cases, with an increase of six strands required.
4. An increase in the tensile limit at service from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ resulted in an increase in maximum span lengths of between approximately 2.5 and 4 ft. for the same number of strands and girder spacing.

8.2.3.2 Trends for Type IV Girders

1. Results showed that for shorter spans (in several cases up to 110 ft., and 120 ft. in one case) an increase in the tensile stress from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ tended to lower the ultimate strength of the beams because this increase resulted in a reduction of the required number of strands for a given span length.
2. For the same span length, an increase in the tensile stress from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ resulted in between two and eight fewer strands required. However, no reductions in the required concrete strength at release (f'_{ci}) were found.
3. An increase in the tensile stress limit at service from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ impacted maximum span lengths for all girder spacings where the tensile stress limit controls. An increase in the tensile limit at service resulted in an increase in maximum span lengths up to 2.1 percent (3.6 percent for one case); however, an increase of two strands was required.
4. An increase in the tensile limit at service from $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$ resulted in an increase in maximum span lengths of between 2 and 4 ft. for the same number of strands and girder spacing.

8.3 RECOMMENDATIONS FOR FLEXURAL DESIGN OF HSC PRESTRESSED GIRDERS

1. In general, for Type IV and U54 girders, designs should consider 0.6 in. diameter strands to effectively use larger concrete strengths (12000 psi), whether they are designed under the Standard or LRFD Specifications. For the same prestressing force, the use of 0.6 in. diameter strands led to a number of strands less than the number required if 0.5 in. diameter strands were used. Consequently, larger prestressing forces could be developed to more fully utilize HSC.
2. For Type IV and U54 girders, considering a transfer length of 60 strand diameters in design can provide additional span capability.
3. For Type IV and U54 girders, the use of the upper limit for the release tensile stress at the beam ends ($7.5\sqrt{f'_{ci}}$ for the Standard Specifications and $6.96\sqrt{f'_{ci}}$ for the LRFD Specifications) rather than the lower limit of $3\sqrt{f'_{ci}}$ leads to a significant increase in the number of strands that can be used, and consequently, increased maximum span lengths can be achieved. This approach is consistent with current TxDOT practices. The parametric study indicated increases in span lengths up to 13 percent for girder spacings less than or equal to 11.5 ft. and up to 30 percent for girder spacings greater than 11.5 ft. for U54 Beams using 0.6 in. diameter strands.
4. The LRFD live load distribution factor equations (for typical cross section "c," referred to as cast-in-place concrete slab on open precast concrete boxes) used in this study for U54 girder spacings less than 11.5 ft. resulted in a significant reduction in the live load moment demand compared with those obtained using the simplified expression *S*/11 for designs under the Standard Specifications. Therefore, this refined distribution factor expression is suggested for use in LRFD designs for U54 Beams.

8.4 RECOMMENDATIONS FOR FUTURE WORK

Based on the findings from this study, the following recommendations are made for future studies.

1. This study focuses on AASHTO Type IV and Texas U54 prestressed concrete bridge girders. It would also be useful to evaluate the impact of LRFD Specifications on other types of bridge girders, as well as to evaluate the potential benefit of using HSC for other types of bridge girders.
2. A preliminary assessment was made to determine the benefit of increasing the tensile stress limit at service from the current value of $6\sqrt{f'_c}$ to $7.5\sqrt{f'_c}$. More studies are recommended to evaluate an increase in the allowable tensile stress at service conditions for prestressed bridge girders. Objections have been raised as to increasing the tensile stress limit because structures under service conditions may be subjected to overloads, resulting in cracking of the concrete, since allowable stresses are specified on a more or less empirical basis. It should be noted that the ACI 318-02 Building Code (ACI 318 2002) now allows design of prestressed members that are allowed to crack under service condition. On the other hand, selection of allowable tensile stresses may include considerations such as the type of girder, strength of concrete (NSC or HSC), and the amount of prestressing and nonprestressing reinforcement. Concerns associated with allowing an increase in the tensile stress include that this limit would tend to lower the ultimate strength of the girder and to increase deflections under overloads due to reduced area of prestressing steel for some cases. Also, cracking may expose the prestressing steel to corrosion and, consequently, to possible fatigue failure. The acceptability of a cracked prestressed section in bridge applications must be considered in refining stress limits. The appropriateness of raising this limit is evaluated further in the report documenting Phase 3 of this study (Hueste et al. 2003c).
3. HSC prestressed bridge girder designs are often controlled by the compressive stress limits. The current limits for the compressive stresses were established to limit excessive creep, camber, or other local strains. Evaluation of the appropriate compressive stress limits for HSC was not within the scope of this project. Because the current code limits are based on NSC properties and because they were found to be critical design criteria,

additional study would be useful to further assess whether these limits should be modified for HSC prestressed girders

4. Safety and serviceability are controlled not only through the use of safety factors but also by statistically defining specified loads and material properties and developing limits that assure an acceptably low probability of failure of exceeding a limit state. The AASHTO LRFD Specifications were developed using reliability theory to calibrate the load and resistance factors for strength limit states. Service limit states should also be evaluated using reliability analysis to assure more consistent safety margins against exceeding conditions that ensure serviceability.

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APPENDIX A

DISTRIBUTION FACTORS AND LIVE LOAD MOMENTS

Table A.1 Comparison of Distribution Factors and Live Load Moments for U54 Beams.

L (ft.)	*S* (ft.)	**Distribution Factors**				**% Dif wrt STD**	**LL Moment/Lane** (k-ft/lane)			**% Dif wrt STD**	**LL Moment per Beam** (k-ft)			**% Dif wrt STD**	**% Dif wrt STD** (consid. S/11)	**% Diff using *S*/11**
		STANDARD		**LRFD**			**STANDARD**		**LRFD**		**STANDARD**	**LRFD**	**LRFD** (consid. *S*/11)			
		DF	Impact	DF	Impact		**Truck**	**Lane**								
90	8.5	0.900	0.233	0.613	0.330	-31.9	1339.8	1053.0	1987.8	48.4%	1486.2	1620.1	2379.4	9.0	60.1	51.1
90	10.0	0.909	0.233	0.689	0.330	-24.2	1339.8	1053.0	1987.8	48.4%	1501.3	1822.7	2403.4	21.4	60.1	38.7
90	11.5	1.045	0.233	0.763	0.330	-27.0	1339.8	1053.0	1987.8	48.4%	1726.4	2017.1	2763.9	16.8	60.1	43.3
90	14.0	1.273	0.233	1.214	0.330	-4.6	1339.8	1053.0	1987.8	48.4%	2101.8	3209.5	3364.8	52.7	60.1	7.4
90	16.6	1.509	0.233	1.439	0.330	-4.6	1339.8	1053.0	1987.8	48.4%	2492.1	3804.4	3989.7	52.7	60.1	7.4
100	8.5	0.900	0.222	0.597	0.330	-33.7	1520.0	1250.0	2320.0	52.6%	1672.0	1841.7	2777.0	10.1	66.1	55.9
100	10.0	0.909	0.222	0.672	0.330	-26.1	1520.0	1250.0	2320.0	52.6%	1688.9	2072.0	2805.1	22.7	66.1	43.4
100	11.5	1.045	0.222	0.743	0.330	-28.9	1520.0	1250.0	2320.0	52.6%	1942.2	2292.9	3225.9	18.1	66.1	48.0
100	14.0	1.273	0.222	1.214	0.330	-4.6	1520.0	1250.0	2320.0	52.6%	2364.4	3745.9	3927.1	58.4	66.1	7.7
100	16.6	1.509	0.222	1.439	0.330	-4.6	1520.0	1250.0	2320.0	52.6%	2803.6	4440.2	4656.5	58.4	66.1	7.7
110	8.5	0.900	0.213	0.583	0.330	-35.2	1699.9	1463.0	2667.9	56.9%	1855.4	2068.0	3193.5	11.5	72.1	60.7
110	10.0	0.909	0.213	0.656	0.330	-27.9	1699.9	1463.0	2667.9	56.9%	1874.1	2326.6	3225.7	24.1	72.1	48.0
110	11.5	1.045	0.213	0.726	0.330	-30.6	1699.9	1463.0	2667.9	56.9%	2155.3	2574.7	3709.6	19.5	72.1	52.7
110	14.0	1.273	0.213	1.214	0.330	-4.6	1699.9	1463.0	2667.9	56.9%	2623.8	4307.6	4516.0	64.2	72.1	7.9
110	16.6	1.509	0.213	1.439	0.330	-4.6	1699.9	1463.0	2667.9	56.9%	3111.1	5106.0	5354.7	64.1	72.1	8.0
120	8.5	0.900	0.204	0.570	0.330	-36.6	1879.7	1692.0	3031.7	61.3%	2037.0	2299.4	3628.9	12.9	78.2	65.3
120	10.0	0.909	0.204	0.642	0.330	-29.4	1879.7	1692.0	3031.7	61.3%	2057.5	2586.9	3665.6	25.7	78.2	52.4
120	11.5	1.045	0.204	0.710	0.330	-32.1	1879.7	1692.0	3031.7	61.3%	2366.2	2862.8	4215.4	21.0	78.2	57.2
120	14.0	1.273	0.204	1.214	0.330	-4.6	1879.7	1692.0	3031.7	61.3%	2880.5	4895.0	5131.8	69.9	78.2	8.2
120	16.6	1.509	0.204	1.439	0.330	-4.6	1879.7	1692.0	3031.7	61.3%	3415.5	5802.2	6084.9	69.9	78.2	8.3
130	8.5	0.900	0.196	0.559	0.330	-37.9	2059.6	1937.0	3411.6	65.6%	2217.1	2536.3	4083.7	14.4	84.2	69.8
130	10.0	0.909	0.196	0.629	0.330	-30.8	2059.6	1937.0	3411.6	65.6%	2239.5	2853.5	4125.0	27.4	84.2	56.8
130	11.5	1.045	0.196	0.696	0.330	-33.4	2059.6	1937.0	3411.6	65.6%	2575.5	3157.8	4743.7	22.6	84.2	61.6
130	14.0	1.273	0.196	1.214	0.330	-4.6	2059.6	1937.0	3411.6	65.6%	3135.4	5508.5	5775.0	75.7	84.2	8.5
130	16.6	1.509	0.196	1.439	0.330	-4.6	2059.6	1937.0	3411.6	65.6%	3717.6	6529.4	6847.5	75.6	84.2	8.6
140	8.5	0.900	0.189	0.549	0.330	-39.0	2240.0	2198.0	3808.0	70.0%	2396.4	2779.0	4558.2	16.0	90.2	74.2
140	10.0	0.909	0.189	0.617	0.330	-32.1	2240.0	2198.0	3808.0	70.0%	2420.6	3126.5	4604.2	29.2	90.2	61.0
140	11.5	1.045	0.189	0.683	0.330	-34.7	2240.0	2198.0	3808.0	70.0%	2783.7	3459.9	5294.9	24.3	90.2	65.9
140	14.0	1.273	0.189	1.214	0.330	-4.6	2240.0	2198.0	3808.0	70.0%	3388.8	6148.5	6445.9	81.4	90.2	8.8
140	16.6	1.509	0.189	1.439	0.330	-4.6	2240.0	2198.0	3808.0	70.0%	4018.2	7288.0	7643.0	81.4	90.2	8.8

Table A.2 Comparison of Distribution Factors and Live Load Moments for Type IV Beams.

L	*S*	**STANDARD**		**LRFD** (D.F. / Impact)							**LL Moment** (k-ft/lane)		**LL Moment per Beam** (k-ft)						
(ft.)	(ft.)	DF	Impact	6000 (psi)	8000 (psi)	10000 (psi)	12000 (psi)	Impact	**% Dif wrt STD**		**STANDARD**	**LRFD**	**STANDARD**	**LRFD**				**% Dif wrt STD**	
		k_g const.==>		1386446	1600930	1789894	1960731		Min (6000)	Max (12000)	**Truck**			6000 (psi)	8000 (psi)	10000 (psi)	12000 (psi)	Min (6000)	Max (12000)
90	4.25	0.386	0.23	0.442	0.448	0.452	0.455	0.33	15	18	1339.8	1987.8	638.0	1169.6	1183.7	1194.8	1203.9	83	89
90	5	0.455	0.23	0.493	0.499	0.504	0.508	0.33	9	12	1339.8	1987.8	750.6	1304.5	1320.5	1333.1	1343.5	74	79
90	5.75	0.523	0.23	0.543	0.550	0.555	0.559	0.33	4	7	1339.8	1987.8	863.2	1435.4	1453.3	1467.4	1479.0	66	71
90	7	0.636	0.23	0.623	0.631	0.637	0.642	0.33	-2	1	1339.8	1987.8	1050.9	1646.2	1667.2	1683.6	1697.3	57	62
90	8.3	0.755	0.23	0.703	0.712	0.719	0.725	0.33	-7	-4	1339.8	1987.8	1246.0	1857.6	1881.6	1900.5	1916.1	49	54
100	4.25	0.386	0.22	0.431	0.436	0.440	0.444	0.33	12	15	1520	2320	717.8	1329.8	1345.7	1358.2	1368.6	85	91
100	5	0.455	0.22	0.480	0.486	0.491	0.495	0.33	6	9	1520	2320	844.4	1482.3	1500.5	1514.7	1526.4	76	81
100	5.75	0.523	0.22	0.528	0.535	0.540	0.544	0.33	1	4	1520	2320	971.1	1630.3	1650.6	1666.5	1679.6	68	73
100	7	0.636	0.22	0.606	0.613	0.619	0.624	0.33	-5	-2	1520	2320	1182.2	1868.7	1892.4	1911.1	1926.5	58	63
100	8.3	0.755	0.22	0.683	0.692	0.699	0.705	0.33	-9	-7	1520	2320	1401.8	2107.8	2135.0	2156.3	2173.9	50	55
110	4.25	0.386	0.21	0.421	0.426	0.430	0.433	0.33	9	12	1699.9	2667.9	796.5	1493.6	1511.4	1525.4	1536.9	88	93
110	5	0.455	0.21	0.469	0.475	0.479	0.483	0.33	3	6	1699.9	2667.9	937.1	1664.1	1684.3	1700.2	1713.4	78	83
110	5.75	0.523	0.21	0.516	0.522	0.527	0.531	0.33	-1	2	1699.9	2667.9	1077.6	1829.4	1852.1	1869.9	1884.6	70	75
110	7	0.636	0.21	0.591	0.598	0.604	0.609	0.33	-7	-4	1699.9	2667.9	1311.9	2095.9	2122.4	2143.2	2160.4	60	65
110	8.3	0.755	0.21	0.666	0.675	0.681	0.687	0.33	-12	-9	1699.9	2667.9	1555.5	2363.0	2393.4	2417.2	2436.9	52	57
120	4.25	0.386	0.20	0.412	0.417	0.421	0.424	0.33	7	10	1879.7	3031.7	874.5	1661.4	1681.0	1696.5	1709.3	90	95
120	5	0.455	0.20	0.459	0.464	0.469	0.472	0.33	1	4	1879.7	3031.7	1028.8	1850.0	1872.5	1890.1	1904.6	80	85
120	5.75	0.523	0.20	0.504	0.510	0.515	0.519	0.33	-4	-1	1879.7	3031.7	1183.1	2033.1	2058.2	2077.9	2094.2	72	77
120	7	0.636	0.20	0.577	0.585	0.590	0.595	0.33	-9	-6	1879.7	3031.7	1440.3	2328.1	2357.4	2380.5	2399.5	62	67
120	8.3	0.755	0.20	0.651	0.659	0.666	0.671	0.33	-14	-11	1879.7	3031.7	1707.8	2623.8	2657.5	2683.9	2705.7	54	58
130	4.25	0.386	0.20	0.404	0.409	0.413	0.416	0.33	5	8	2059.6	3411.6	951.8	1833.3	1854.9	1871.9	1885.9	93	98
130	5	0.455	0.20	0.450	0.455	0.459	0.463	0.33	-1	2	2059.6	3411.6	1119.8	2040.6	2065.2	2084.6	2100.5	82	88
130	5.75	0.523	0.20	0.494	0.500	0.505	0.509	0.33	-5	-3	2059.6	3411.6	1287.7	2241.7	2269.3	2290.9	2308.8	74	79
130	7	0.636	0.20	0.565	0.573	0.578	0.583	0.33	-11	-8	2059.6	3411.6	1567.7	2565.8	2598.0	2623.4	2644.3	64	69
130	8.3	0.755	0.20	0.637	0.645	0.652	0.657	0.33	-16	-13	2059.6	3411.6	1858.8	2890.7	2927.6	2956.7	2980.6	56	60
140	4.25	0.386	0.19	0.397	0.401	0.405	0.408	0.33	3	6	2240	3808	1028.7	2009.6	2033.3	2051.8	2067.1	95	101
140	5	0.455	0.19	0.441	0.447	0.451	0.454	0.33	-3	0	2240	3808	1210.3	2235.9	2262.8	2283.9	2301.4	85	90
140	5.75	0.523	0.19	0.485	0.491	0.495	0.499	0.33	-7	-4	2240	3808	1391.8	2455.5	2485.6	2509.2	2528.7	76	82
140	7	0.636	0.19	0.555	0.562	0.567	0.572	0.33	-13	-10	2240	3808	1694.4	2809.3	2844.4	2872.1	2894.9	66	71
140	8.3	0.755	0.19	0.625	0.633	0.639	0.644	0.33	-17	-15	2240	3808	2009.1	3163.9	3204.3	3236.0	3262.1	57	62

APPENDIX B

SURVEY OF CURRENT PRACTICE

Texas Department of Transportation Project 0-2101
"Allowable Stresses and Resistance Factors for High Strength Concrete"

Questionnaire
Current Practice for Design of High Strength Concrete Prestressed Members

Overview and Instructions

Thank you for taking the time to respond to the enclosed questionnaire. This information is being collected as part of a TxDOT research project to develop a review of the current state of practice for the design of high strength concrete (HSC) prestressed bridge girders. The definition of HSC for this study is *concrete with specified compressive strengths for design of 6000 psi or greater, made without using exotic materials or techniques*. This is consistent with the working definition for HSC given by ACI Committee 363 in their state-of-the-art report.

The questionnaire consists of two parts. If a particular question is not applicable, please note as such. Please answer the following questions as completely as possible, using additional pages as needed, and return to the address below by *Friday, June 14, 2002*. We appreciate your input.

Please mail or fax questionnaire to:

Mary Beth Hueste, Ph.D., P.E.
Texas Transportation Institute
3135 TAMU
Texas A&M University System
College Station, Texas 77843-3135
Phone: (979) 845-1940
Fax: (979) 845-6554
E-mail: mhueste@tamu.edu

Contact Information *(person answering this questionnaire)*

Name	
Organization	
Address	
Phone	
Fax	
E-mail	

Do we have permission to identify your organization, as appropriate, when reporting the responses to this questionnaire?

❒ Yes ❒ No Signature: ______________________________

Part I: Current Design Practice for HSC Prestressed Bridge Members

Question	Answer
1. Current specification used by your organization for bridge member design.	❒ AASHTO LRFD Specifications ❒ AASHTO Standard Specifications Specification Year: ________________
2. If your organization is currently using the AASHTO LRFD Specifications, when were they implemented in your state (provide year)?	
3. If your organization plans to use the AASHTO LRFD Specifications in the future, when do you foresee their implementation in your state (provide year)?	
4. Please list any other documents used by your organization for the design of *prestressed concrete bridge girders*.	
5. Please list any additional reference documents used by your organization for design of *HSC members*.	
6. Please provide the names and locations of precasters that supply HSC prestressed girders for your bridge projects.	
7. How many bridges does your organization typically construct each year? Of these, what percentage use HSC prestressed bridge girders (specified $f'_c > 6000$ psi)?	Number per year: _________________ Percentage HSC: __________________

Question	Answer
8. Please provide the typical range of specified strength for prestressed concrete bridge girders used in current projects for your organization.	Range of specified design concrete compressive strength at *transfer* (f'_{ci}): ____________________ psi Range of specified design concrete compressive strength in *service* (f'_c): ____________________ psi
9. Please comment on whether the need to meet the required concrete compressive strength at transfer (f'_{ci}) in a short period of time has led to a practice where precasters use mix designs that give significantly a larger value of f'_c in service than specified. If this practice has been observed by your organization, can you give any specific information as to how this overstrength varies as a function of specified f'_{ci} and f'_c values?	
10. Please note any concerns you have related to the use of HSC prestressed bridge girders.	
11. Has your organization made any adjustments to the design specifications for HSC prestressed bridge girders based on research findings (such as in the allowable stresses or resistance factors)? If so, please describe and provide a reference to relevant research, if available.	

Part II: Description of Typical Bridges with HSC Prestressed Bridge Members

In the following table, please provide the following information based on the practices of your organization.

- Indicate the types of bridges for which HSC prestressed bridge girders have been used by your organization.
- Provide the ranges for span length and concrete compressive strength (f'_c), for each structural type selected.
- Note how prevalent each type is for HSC prestressed bridge members, by filling in the percentage column.

Span Type	Structural Type	Span (range in ft.)	f'_c (range in psi)	Percentage
Simple Span	Slab			
	Voided Slab			
	Double T			
	Closed Box CIP			
	AASHTO Beam			
	Bulb			
	Box Girder			
	Other (describe)			
Continuous Span*	Slab			
	Voided Slab			
	AASHTO Beam			
	Post-tensioned AASHTO Beam			
	Bulb			
	Box			
	Other (describe)			

* For this study, the term "continuous span" refers to the case where the girders are continuous over a support. When continuity is provided within the cast-in-place slab only, this is considered a "simple span".

Table B.1. Precasters per DOT (Q 6).

Department of Transportation	Precasters	
	Name	Location
Alabama (AL)	Gulf Coast Prestress	Pass Christian. MS
	Sherman Prestress	Pelham, AL
	Standard Prestress	Atlanta, GA
	Tindal Prestress	Atlanta, GA
Alaska (AK)	Aggregate Products Inc.	Anchorage, AK
	Concrete Technology	Taloma, WA
Arkansas (AR)	N/A	N/A
Colorado (CO)	Alun Creek Structures	Littleton, CO
	Hydro Conduit	Denver, CO
	Rocky Mountain Prestress	Denver, CO
Connecticut (CT)	-	-
Florida (FL)	Dura-Stress, Inc.	Leesburg, FL
	Gate Concrete Products Company	Jacksonville, FL
	Gulf Coast Prestress	Pass Christian, MS
	Standard Concrete Products	Tampa, FL
	Standard Concrete Products	Savannah, GA
Georgia (GA)	Standard Concrete Products	Atlanta, GA
	Tondall Corporation	Atlanta, GA
Hawaii (HI)	Hawaiian Bitumuls Paving Precast Co.	Kapolei, HI
	Rocky Mountain Prestress	Kapolei, HI
Idaho (ID)	Central Premix Prestress Company	Spokane, WA
	Eagle Precast Company	Eagle, ID
	Teton Prestress Concrete LLC	Idaho Falls, ID
Illinois (IL)	Prestressed Eng. (PEC)	Prairie Grove, IL
Iowa (IA)	Andrews Prestress Concrete	Clear Lute, IA
	CSR Wilson	Bellevue, NE
	Humbolt Concrete Products	Humbolt, IA
	Iowa Concrete Products	Iowa Falls, IA
	Raider Concrete	Burlington, IA
Kansas (KS)	Prestressed Concrete Inc.	Newton, KS
	Rinker Materials	Kansas City, MO
Kentucky (KY)	Hydro Conduit	Henderson, KY
	Prestressed Services	Lexington, KY
Louisiana (LA)	Gulf Coast Prestress	Pass Christian, MS
Massachusetts (MA)	Brakeslee Prestressed, Inc.	Brandford, CT
	J.P. Carrera & Sons	Middlebury, VT
	Northeast Concrete Products	MA
	Old Castle Precast, Inc.	Bethleham, NY
	Strescon Limited	NB
	Unistress Corp.	Pittsfields, MA

Table B.1. Continued.

Department of Transportation	Precasters	
	Name	Location
Michigan (MI)	Premare	Grand Rapids, MI
	Stresscon	Bay City, MI
Minnesota (MN)	Andrews Prestress Concrete	Mason City, IA
	County Concrete	Osseo, MN
	Elk River Concrete	Elk River, MN
Mississippi (MS)	Gulf Coast Prestresss	Pass Christian, MS
	J.J. Fergnsa	Greenwood, MS
	Madison Materials	Ridgeland, MS
Missouri (MO)	CSR/Wilson	Kansas City, KS
	Egyptian Concrete	Bonne Terre, MO
	Rinker Materials	Marshall, MO
Montana (MT)	Central Premix Prestress Company	Spokane, WA
	Montana Prestressed Concrete	Helena, MT
Nevada (NV)	N/A	N/A
New Hampshire (NH)	J.P. Carrera & Sons	Middleburg, VT
	Northeast Concrete Products	Plainville, MA
	Unistress Corp.	Pittsfield, MA
New Jersey (NJ)	Bayshore Concrete Products Corp.	Cape Charles, VA
	Precast System, Inc.	Lakewood, NJ
New Mexico (NM)	Rinker Materials	Albuquerque, NM
New York (NY)		
North Carolina (NC)	Bayshore Concrete Products	Chesapeake, VA
	Carolina Prestress	Charlotte, NC
	Florence Concrete Products	Sunter, SC
	Ross Presstress Concrete	Bristol, TN
	S&G Prestress	Wilmington, NC
	Standard Concrete Products	Atlanta, GA
	Utility Precast	Charlote, NC
North Dakota (ND)	North Dakota Concrete Products	Bismarck, ND
Ohio (OH)	PSI	Decator, IN
	PSI	Grove City, OH
	PSI	Melbourne, KY
	United Precast	Mt. Vernon, OH
Oklahoma (OK)	Bexar Concrete Works	San Antonio, TX
	Rinker (Hydro)	
Pennsylvania (PA)	Hank Bonstadt	Not available
	PCAP	Not available
Rhode Island (RI)	Northeast Concrete Products	Plainville, MA
	Rotondo Precast	Rehoboth, MA
South Carolina (SC)	Florence Concrete Products	SC
	Standard Concrete Products	SC

Table B.1. Continued.

Department of Transportation	Precasters	
	Name	Location
South Dakota (SD)	Cretex	Elk River, MN
	Gage Bros.	Sioux Falls, SD
	S.D. Concrete Products	Rapid City, SD
Tennessee (TN)	Construction Products Inc.	Jackson, TN
	CPI Concrete Products	Memphis, TN
	Ross Prestressed Concrete	Knoxville TN
Texas - Austin (TX)	Bexar Concrete Works	San Antonio, TX
	Flexicore	Houston, TX
	Heldenfels	San Marcos, TX
	MANCO	San Antonio, TX
	Southwest Prestressed	Amarillo, TX
	Texas Concrete	Victoria, TX
	Texas Prestressed	Elm Mott, TX
Vermont (VT)	J.P. Carrera & Sons	Middlebury, VT
	Old Castle Precast, Inc.	Bethlehem, NY
Virginia (VA)	Bayshore	VA
	Rotondo Precast	VA
Washington (WA)	Central Premix Prestress Company	Spokane, WA
	Concrete Technology	Tacoma, WA
	Morse Brothers	Eugene, OR
Wisconsin (WI)	County Concrete	Eau Claire, WI
	Spancrete	Green Bay, WI
Arizona (AZ)		
Additional Respondents		
Texas - Houston (TX)	Bexar Concrete Works	San Antonio, TX
	Heldenfels	Corpus/San Marcos, TX
	Texas Concrete	Victoria, TX
Structural Engineering Associates	Bexar Concrete Works	San Antonio, TX
	Flexicore	Houston, TX
	Heldenfels	San Marcos, TX
	Manufactured Concrete, Ltd.	San Antonio, TX
	Texas Concrete	Victoria, TX
	Texas Prestress	Waco, TX
Turner, Collie & Braden, Inc.	Texas Concrete	Victoria, TX

Table B.2. Precasters and Supplied DOTs (Q 6).

Precaster	Precaster's Location by State	States Supplied by Precaster
Aggregate Products Inc.	Anchorage, AK	Alaska
Alun Creek Structures	Littleton, CO	Colorado
Andrews Prestressed Concrete	Clear Lake, IA	Iowa
	Mason City, IA	Minnesota
Bayshore Concrete Products	VA	Virginia
	Chesapeake, VA	North Carolina
	Cape Charles, VA	New Jersey
Bexar Concrete Works	San Antonio, TX	Oklahoma
		Texas - Austin
		Texas - Houston
		Structural Eng. Assoc.
Brakeslee Prestressed, Inc.	Not available	Massachusetts
Carolina Prestress	Charlotte, NC	North Carolina
Central Premix Prestress Co.	Spokane, WA	Idaho
		Montana
		Washington
Concrete Technology	Tacoma, WA	Alaska
		Washington
Construction Products Inc.	Jackson, TN	Tennessee
County Concrete	Osseo, MN	Minnesota
	Eau Claire, WI	Wisconsin
CPI Concrete Products	Memphis, TN	Tennessee
Cretex	Elk River, MN	South Dakota
CSR/Wilson	Kansas City, KS	Missouri
	Bellevue, NE	Iowa
Dura-Stress, Inc.	Leesburg, FL	Florida
Eagle Precast Company	Eagle, ID	Idaho
Egyptian Concrete	Bonne Terre, MO	Missouri
Elk River Concrete	Elk River, MN	Minnesota
Flexicore	Houston, TX	Texas - Austin
		Structural Eng. Assoc.
Florence Concrete Products	Sunter, SC	North Carolina
		South Carolina
Gage Bros.	Sioux Falls, SD	South Dakota
Gate Concrete Products Co.	Jacksonville, FL	Florida
Gulf Coast Prestress	Pass Christian, MS	Alabama
		Louisiana
		Florida
		Mississippi
Hank Bonstadt	Not available	Pennsylvania

Table B.2. Continued.

Precaster	Precaster's Location By State	States Supplied by Precaster
Hawaiian Bitumuls Paving Prec.	Kapolei, HI	Hawaii
Heldenfels	San Marcos, TX	Texas - Austin
		Structural Eng. Assoc.
	Corpus Christi, TX	Texas - Houston
	San Marcos, TX	
Humbolt Concrete Products	Humbolt, IA	Iowa
Hydro Conduit	Denver, CO	Colorado
	Henderson, KY	Kentucky
Iowa Concrete Products	Iowa Falls, IA	Iowa
J.J. Fergnsa	Greenwood, MS	Mississippi
J.P. Carrera & Sons	Middlebury, VT	Massachusetts
		New Hampshire
		Vermont
Madison Materials	Ridgeland, MS	Mississippi
MANCO	San Antonio, TX	Texas - Austin
Manufactured Concrete, Ltd.	San Antonio, TX	Structural Eng. Assoc.
Montana Prestressed Con.	Helena, MT	Montana
Morse Brothers	Eugene, OR	Washington
N/A	N/A	Arkansas
N/A	N/A	Nevada
North Dakota Concrete Prod.	Bismarck, ND	North Dakota
Northeast Concrete Prod.	Plainville, MA	Massachusetts
		New Hampshire
		Rhode Island
Old Castle Precast, Inc.	Bethlehem, NY	Vermont
		Massachusetts
PCAP	Not available	Pennsylvania
Precast System, Inc.	Lakewood, NJ	New Jersey
Premare	Grand Rapids, MI	Michigan
Prestressed Concrete Inc.	Newton, KS	Kansas
Prestressed Eng. (PEC)	Prairie Grove, IL	Illinois
Prestressed Services	Lexington, KY	Kentucky
PSI	Decator, IN	Ohio
	Grove City, OH	
	Melbourne, KY	
Raider Concrete	Burlington, IA	Iowa
Rinker (Hydro)	Tulsa, OK & Oklahoma City, OK	Oklahoma
Rinker Materials	Kansas City, MO	Kansas
	Marshall, MO	Missouri
	Albuquerque, NM	New Mexico
Rocky Mountain Prestress	Denver, CO	Colorado
	Kapolei, HI	Hawaii

Table B.2. Continued.

Precaster	Precaster's Location by State	States Supplied by Precaster
Ross Prestressed Concrete	Bristol, TN	North Carolina
	Knoxville, TN	Tennessee
Rotondo Precast	Rehoboth, MA	Rhode Island
	Not Available	Virginia
S&G Prestress	Wilmington, NC	North Carolina
S.D. Concrete Products	Rapid City, SD	South Dakota
Sherman Prestress	Pelham, AL	Alabama
Southwest Prestressed	Amarillo, TX	Texas - Austin
Spancrete	Green Bay, WI	Wisconsin
Standard Concrete Products	Tampa, FL	Florida
	Savannah, GA	Florida
	Atlanta, GA	Georgia
	Atlanta, GA	North Carolina
	SC	South Carolina
Standard Prestress	Atlanta, GA	Alabama
Strescon	Bay City, MI	Michigan
Strescon Limited	NB	Massachusetts
Teton Prestress Concrete	Idaho Falls, ID	Idaho
Texas Concrete	Victoria, TX	Texas - Austin
		Texas - Houston
		Structural Eng. Assoc.
		Turner, Collie & Braden
	Waco, TX	Structural Eng. Assoc.
Texas Prestress	Elm Mott, TX	Texas - Austin
Tindal Prestress	Atlanta, GA	Alabama
Tondall Corporation	Atlanta, GA	Georgia
Unistress Corp.	Pittsfield, MA	New Hampshire
	Pittsfield, MA	Massachusetts
United Precast	Mt. Vernon, OH	Ohio
Utility Precast	Charlotte, NC	North Carolina

APPENDIX C

RESULTS FOR U54 BEAMS

Note: For a description of the controlling limit states, refer to Table 6.2.

Table C.1. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	32	4.528	12.469	6163	6218	Ultimate Moment Strength/f(t)_T L
	4500	100	41	5.422	14.545	7242	7797	f(t) _ Total Load
	4500	110	51	6.388	16.779	8385	9494	f(t) _ Total Load
	4500	114.1	57	7.157	18.433	8876	10463	f(c)_Total Dead Load
8000	6000	90	32	4.085	12.094	6163	6199	Ultimate Moment Strength
	6000	100	39	4.608	13.271	7242	7423	f(t) _ Total Load
	6000	110	50	5.590	15.853	8385	9282	f(t) _ Total Load
	6000	120	62	6.560	18.393	9594	11170	f(t) _ Total Load
	6000	130	80	8.090	22.691	10869	13814	f(c)_Total Dead Load
10000	7500	90	32	3.779	11.835	6163	6199	Ultimate Moment Strength
	7500	100	38	4.133	12.573	7242	7250	f(t) _ Total Load
	7500	110	48	4.908	14.711	8385	8950	f(t) _ Total Load
	7500	120	60	5.806	17.261	9594	10866	f(t) _ Total Load
	7500	130	74	6.804	20.146	10869	12963	f(t) _ Total Load
	7500	140	94	8.064	24.101	12210	15467	f(t) _ Total Load
	7500	140.8	96	8.137	24.376	12316	15642	f(c)_Total Dead Load
12000	9000	90	32	3.550	11.641	6163	6199	Ultimate Moment Strength
	9000	100	38	3.876	12.360	7242	7250	Ultimate Moment Strength
	9000	110	47	4.482	14.068	8385	8783	f(t) _ Total Load
	9000	120	58	5.222	16.249	9594	10559	f(t) _ Total Load
	9000	130	72	6.147	19.118	10869	12673	f(t) _ Total Load
	9000	140	90	7.262	22.746	12210	15082	f(t) _ Total Load
	9000	142.8	99	7.610	24.164	12600	15902	f(t) _ Total Load

Table C.2. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 10 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	34	4.839	13.043	6393	6629	f(t) _ Total Load
	4500	100	44	5.872	15.446	7522	8405	f(t) _ Total Load
	4500	110	55	6.959	17.953	8721	10291	f(t) _ Total Load
	4500	110.2	55	6.954	17.935	8732	10291	f(c)_Total Dead Load
8000	6000	90	33	4.225	12.221	6393	6431	Ultimate Moment Strength
	6000	100	42	5.005	14.143	7522	8026	f(t) _ Total Load
	6000	110	53	5.977	16.615	8721	9915	f(t) _ Total Load
	6000	120	67	7.136	19.701	9990	12136	f(t) _ Total Load
	6000	125.3	75	7.779	21.409	10690	13355	f(c)_Total Dead Load
10000	7500	90	33	3.905	11.951	6393	6431	Ultimate Moment Strength
	7500	100	40	4.375	13.020	7522	7676	Ultimate Moment Strength/f(t)_TL
	7500	110	51	5.259	15.456	8721	9576	f(t) _ Total Load
	7500	120	65	6.331	18.549	9990	11826	f(t) _ Total Load
	7500	130	80	7.412	21.637	11330	14101	f(t) _ Total Load
	7500	136.5	95	8.280	24.454	12241	15985	f(t) _ Total Load / f(c)_T D L
12000	9000	90	33	3.666	11.750	6393	6431	Ultimate Moment Strength
	9000	100	39	3.992	12.398	7522	7500	f(t) _ Total Load
	9000	110	50	4.806	14.797	8721	9406	f(t) _ Total Load
	9000	120	63	5.707	17.523	9990	11514	f(t) _ Total Load
	9000	130	78	6.710	20.595	11330	13804	f(t) _ Total Load
	9000	138.8	99	7.775	24.381	12566	16344	f(t) _ Total Load

Table C.3. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 11.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	38	5.445	14.462	7079	7398	f(t) Total Load
	4500	100	49	6.605	17.154	8316	9358	f(t) Total Load
	4500	106.9	58	7.512	19.307	9211	10893	f(c) Total Dead Load
8000	6000	90	37	4.756	13.590	7079	7198	Ultimate Moment Strength
	6000	100	47	5.651	15.803	8316	8975	f(t) Total Load
	6000	110	60	6.799	18.788	9626	11174	f(t) Total Load
	6000	120	76	8.139	22.377	11011	13712	f(t) Total Load
	6000	121.7	80	8.497	23.398	11250	14328	f(c) Total Dead Load
10000	7500	90	37	4.382	13.279	7079	7198	Ultimate Moment Strength
	7500	100	46	5.075	15.042	8316	8800	f(t) Total Load
	7500	110	58	6.021	17.638	9626	10848	f(t) Total Load
	7500	120	74	7.244	21.193	11011	13401	f(t) Total Load
	7500	130	96	8.600	25.461	12471	16378	f(t) Total Load
	7500	130.9	99	8.691	25.842	12606	16686	f(t) Total Load
12000	9000	90	37	4.105	13.047	7079	7198	Ultimate Moment Strength
	9000	100	44	4.530	13.999	8316	8447	Ultimate Moment Strength / f(t)_TL
	9000	110	57	5.515	16.968	9626	10684	f(t) Total Load
	9000	120	72	6.552	20.145	11011	13088	f(t) Total Load
	9000	130	92	7.792	24.183	12471	15952	f(t) Total Load
	9000	132.2	99	8.025	25.167	12798	16686	f(t) Total Load

Table C.4. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 14 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	46	6.625	17.312	8223	8919	f(t) Total Load
	4714	100	60	7.951	20.708	9639	11373	f(t) Total Load
	4925	102.2	63	8.084	21.253	9958	11878	f(c) Total Dead Load
8000	6000	90	44	5.666	15.948	8223	8534	f(t) Total Load
	6000	100	57	6.881	19.071	9639	10828	f(t) Total Load
	6000	110	74	8.362	23.046	11135	13639	f(t) Total Load
	6456	115.9	85	8.987	25.264	12059	15371	f(c) Total Dead Load
10000	7500	90	43	5.086	15.180	8223	8354	Ultimate Moment Strength
	7500	100	55	6.094	17.909	9639	10490	f(t) Total Load
	7500	110	71	7.345	21.501	11135	13151	f(t) Total Load
	7500	120	93	8.849	26.097	12713	16432	f(t) Total Load
	7500	121.5	99	9.052	26.929	12957	17071	f(t) Total Load
12000	9000	90	43	4.748	14.906	8223	8354	Ultimate Moment Strength
	9000	100	54	5.580	17.230	9639	10321	f(t) Total Load
	9000	110	69	6.644	20.443	11135	12824	f(t) Total Load
	9000	120	90	8.049	24.969	12713	16060	f(t) Total Load
	9000	122.7	99	8.361	26.249	13155	17071	f(t) Total Load

Table C.5. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 16.6 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	54	7.772	20.043	9413	10443	f(t) Total Load
	5295	97.8	67	8.505	22.779	10661	12682	f(c) Total Dead Load
8000	6000	90	52	6.678	18.610	9413	10054	f(t) Total Load
	6000	100	69	8.222	22.726	11015	12976	f(t) Total Load
	6882	110	90	9.368	26.972	12704	16313	f(t) Total Load
	6941	110.2	91	9.393	27.155	12734	16447	f(c) Total Dead Load
10000	7500	90	50	5.890	17.402	9413	9692	f(t) Total Load
	7500	100	67	7.313	21.518	11015	12639	f(t) Total Load
	7500	110	87	8.841	25.984	12704	15895	f(t) Total Load
	7500	113.2	99	9.343	27.836	13285	17373	f(t) Total Load
12000	9000	90	49	5.382	16.704	9413	9511	Ultimate Moment Strength/f(t)_TL
	9000	100	65	6.613	20.453	11015	12300	f(t) Total Load
	9000	110	85	8.059	24.980	12704	15997	f(t) Total Load
	9000	114.5	99	8.632	27.151	13490	17373	f(t) Total Load

Table C.6. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strand	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	23	4.683	12.874	6163	6347	Ultimate Moment Strength
	4500	100	28	5.347	14.256	7242	7651	Ultimate Moment Strength/f(t)_TL
	4500	110	36	6.510	17.053	8385	9580	f(t) _ Total Load
	4500	115.1	40	7.055	18.328	8988	10516	f(c)_Total Dead Load
8000	6000	90	23	4.220	12.484	6163	6347	Ultimate Moment Strength
	6000	100	27	4.617	13.220	7242	7377	Ultimate Moment Strength/f(t)_TL
	6000	110	35	5.644	15.948	8385	9299	f(t) _ Total Load
	6000	120	43	6.598	18.393	9594	11139	f(t) _ Total Load
	6000	130	53	7.824	21.646	10869	13349	f(t) _ Total Load
	6000	131.8	55	8.057	22.273	11107	13762	f(c)_Total Dead Load
10000	7500	90	23	3.900	12.214	6163	6347	Ultimate Moment Strength
	7500	100	27	4.258	12.924	7242	7377	Ultimate Moment Strength
	7500	110	34	5.022	15.026	8385	9063	f(t) _ Total Load
	7500	120	42	5.889	17.435	9594	10914	f(t) _ Total Load
	7500	130	51	6.853	20.124	10869	12921	f(t) _ Total Load
	7500	140	63	8.108	23.835	12210	15330	f(t) _ Total Load
	7608	145.5	71	8.887	26.322	12970	16814	f(c)_Total Dead Load
12000	9000	90	23	3.661	12.013	6163	6347	Ultimate Moment Strength
	9000	100	27	3.991	12.704	7242	7377	f(t) _ Total Load
	9000	110	33	4.542	14.210	8385	8826	f(t) _ Total Load
	9000	120	41	5.344	16.594	9594	10687	f(t) _ Total Load
	9000	130	50	6.236	19.260	10869	12704	f(t) _ Total Load
	9000	140	61	7.281	22.501	12210	14955	f(t) _ Total Load
	9000	150	76	8.721	27.190	13617	17746	f(t) _ Total Load
	9432	157	89	9.665	30.894	14638	19832	f(c)_Total Dead Load

Table C.7. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 10 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	24	4.922	13.228	6393	6684	Ultimate Moment Strength
	4500	100	30	5.788	15.097	7522	8227	f(t) _ Total Load
	4500	110	38	6.927	17.779	8721	10187	f(t) _ Total Load
	4500	111.2	40	7.270	18.862	8866	10666	f(c)_Total Dead Load
8000	6000	90	23	4.228	12.186	6393	6401	Ultimate Moment Strength
	6000	100	29	4.996	14.030	7522	7951	f(t) _ Total Load
	6000	110	37	6.013	16.646	8721	9903	f(t) _ Total Load
	6000	120	46	7.131	19.526	9990	12016	f(t) _ Total Load
	6000	127.3	55	8.293	22.687	10966	14033	f(c)_Total Dead Load
10000	7500	90	23	3.908	11.916	6393	6401	Ultimate Moment Strength
	7500	100	28	4.434	13.134	7522	7702	Ultimate Moment Strength / f(t)_T L
	7500	110	36	5.356	15.706	8721	9663	f(t) _ Total Load
	7500	120	45	6.374	18.545	9990	11785	f(t) _ Total Load
	7500	130	55	7.467	21.594	11330	14033	f(t) _ Total Load
	7500	140	69	8.939	25.996	12740	16903	f(t) _ Total Load
	7531	140.5	70	9.037	26.345	12807	17098	f(c)_Total Dead Load
12000	9000	90	23	3.669	11.715	6393	6401	Ultimate Moment Strength
	9000	100	27	4.000	12.336	7522	7452	Ultimate Moment Strength / f(t)_T L
	9000	110	35	4.851	14.877	8721	9421	f(t) _ Total Load
	9000	120	44	5.791	17.687	9990	11554	f(t) _ Total Load
	9000	130	54	6.813	20.740	11330	13822	f(t) _ Total Load
	9000	140	67	8.055	24.657	12740	16516	f(t) _ Total Load
	9000	150	84	9.675	29.942	14222	19706	f(t) _ Total Load
	9375	151.8	88	9.836	30.980	14493	20344	f(c)_Total Dead Load

Table C.8. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 11.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	27	5.617	14.868	7079	7528	Ultimate Moment Strength/f(t)_T L
	4500	100	34	6.615	17.103	8316	9298	f(t) _ Total Load
	4638	108.1	42	7.740	19.965	9369	11265	f(c)_Total Dead Load
8000	6000	90	26	4.837	13.769	7079	7242	Ultimate Moment Strength
	6000	100	33	5.734	15.978	8316	9017	f(t) _ Total Load
	6000	110	42	6.904	18.998	9626	11218	f(t) _ Total Load
	6000	120	53	8.332	22.766	11011	13802	f(t) _ Total Load
	6205	123.5	57	8.679	23.910	11499	14686	f(c)_Total Dead Load
10000	7500	90	26	4.455	13.454	7079	7242	Ultimate Moment Strength
	7500	100	32	5.101	15.044	8316	8768	f(t) _ Total Load
	7500	110	41	6.164	18.016	9626	10977	f(t) _ Total Load
	7500	120	51	7.313	21.216	11011	13341	f(t) _ Total Load
	7500	130	64	8.721	25.334	12471	16180	f(t) _ Total Load
	8008	136.2	74	9.527	28.366	13419	18262	f(c)_Total Dead Load
12000	9000	90	26	4.170	13.218	7079	7242	Ultimate Moment Strength
	9000	100	31	4.614	14.219	8316	8518	f(t) _ Total Load
	9000	110	40	5.595	17.158	9626	10735	f(t) _ Total Load
	9000	120	50	6.658	20.329	11011	13108	f(t) _ Total Load
	9000	130	62	7.849	23.983	12471	15758	f(t) _ Total Load
	9000	140	79	9.537	29.457	14006	19257	f(t) _ Total Load
	9828	146.1	93	10.268	33.107	14970	21593	f(c)_Total Dead Load

Table C.9. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 14 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	32	6.669	17.350	8223	8885	f(t) _ Total Load
	4500	100	42	8.233	21.077	9639	11414	f(t) _ Total Load
	5072	103.5	45	8.252	21.719	10147	12159	f(c)_Total Dead Load
8000	6000	90	31	5.777	16.212	8223	8605	f(t) _ Total Load
	6000	100	40	6.988	19.303	9639	10878	f(t) _ Total Load
	6000	110	51	8.456	23.130	11135	13568	f(t) _ Total Load
	6830	117.9	62	9.315	26.477	12372	16088	f(c)_Total Dead Load
10000	7500	90	30	5.138	15.267	8223	8348	Ultimate Moment Strength / f(t)_T L
	7500	100	39	6.237	18.308	9639	10629	f(t) _ Total Load
	7500	110	49	7.422	21.563	11135	13086	f(t) _ Total Load
	7500	120	63	9.018	26.236	12713	16310	f(t) _ Total Load
	8830	130	81	10.264	31.702	14370	20175	f(c)_Total Dead Load
12000	9000	90	30	4.796	14.991	8223	8348	Ultimate Moment Strength
	9000	100	37	5.515	16.901	9639	10128	f(t) _ Total Load
	9000	110	48	6.757	20.663	11135	12844	f(t) _ Total Load
	9000	120	61	8.122	24.871	12713	15865	f(t) _ Total Load
	9000	130	79	9.946	30.767	14373	19758	f(t) _ Total Load
	10283	136.9	99	10.712	35.472	15560	23106	f(t) _ Total Load

Table C.10. U54 Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 16.6 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	38	7.890	20.298	9413	10501	f(t) _ Total Load
	5609	99.4	49	8.798	23.832	10923	13287	f(c) _ Total Dead Load
8000	6000	90	36	6.679	18.525	9413	9961	f(t) _ Total Load
	6000	100	48	8.371	23.010	11015	12992	f(t) _ Total Load
	6956	110	62	9.570	27.404	12704	16330	f(t) _ Total Load
	7362	112.7	66	9.785	28.501	13181	17243	f(c) _ Total Dead Load
10000	7500	90	35	5.955	17.536	9413	9704	f(t) _ Total Load
	7500	100	46	7.341	21.432	11015	12494	f(t) _ Total Load
	7500	110	60	9.011	26.272	12704	15869	f(t) _ Total Load
	8687	120	79	10.474	32.253	14483	20141	f(t) _ Total Load
	9579	123.7	88	10.845	34.712	15170	21979	f(c) _ Total Dead Load
12000	9000	90	34	5.399	16.672	9413	9446	Ultimate Moment Strength / f(t)_T L
	9000	100	45	6.679	20.527	11015	12244	f(t) _ Total Load
	9000	110	58	8.113	24.898	12704	15406	f(t) _ Total Load
	9000	120	77	10.085	31.280	14483	19702	f(t) _ Total Load
	10355	127.5	99	10.990	36.554	15878	23722	f(t) _ Total Load

Table C.11. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	29	3.977	12.361	5428	5680	f(t) _Total Load
	4500	100	38	4.931	14.410	6420	7277	f(t) _Total Load
	4500	119.6	59	7.031	18.921	8566	10774	f(c) _Total Dead Load
8000	6000	90	28	3.444	11.638	5428	5485	Ultimate Moment Strength/f(t)_T.L.
	6000	100	36	4.146	13.220	6420	6902	f(t) _Total Load
	6000	110	46	5.061	15.368	7483	8615	f(t) _Total Load
	6000	120	58	6.137	17.952	8618	10559	f(t) _Total Load
	6000	130	72	7.309	20.816	9825	12673	f(t) _Total Load
	6018	136.3	82	8.122	22.836	10628	14084	f(c) _Total Dead Load
10000	7500	90	28	3.190	11.437	5428	5485	Ultimate Moment Strength
	7500	100	35	3.696	12.576	6420	6727	f(t) _Total Load
	7500	110	45	4.529	14.682	7483	8446	f(t) _Total Load
	7500	120	56	5.404	16.888	8618	10251	f(t) _Total Load
	7500	130	70	6.479	19.730	9824	12362	f(t) _Total Load
	7500	140	86	7.650	22.856	11104	14609	f(t) _Total Load
	7500	144.6	99	8.310	24.989	11713	15876	f(t) _Total Load
12000	9000	90	28	3.002	11.288	5428	5485	Ultimate Moment Strength
	9000	100	34	3.349	12.008	6420	6552	f(t) _Total Load
	9000	110	43	4.004	13.703	7483	8108	f(t) _Total Load
	9000	120	54	4.832	15.927	8618	9939	f(t) _Total Load
	9000	130	68	5.833	18.758	9824	12067	f(t) _Total Load
	9000	140	84	6.945	21.924	11104	14360	f(t) _Total Load
	9000	145.9	99	7.675	24.377	11889	15902	f(t) _Total Load

Table C.12 U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 10 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	33	4.629	13.772	5944	6449	f(t) _Total Load
	4500	100	42	5.562	15.701	7023	8054	f(t) _Total Load
	4500	110	54	6.873	18.556	8178	10128	f(t) _Total Load
	4547	115.7	61	7.496	19.938	8874	11256	f(c)_Total Dead Load
8000	6000	90	31	3.876	12.591	5944	6071	Ultimate Moment Strength
	6000	100	41	4.844	14.871	7023	7851	f(t) _Total Load
	6000	110	52	5.874	17.272	8178	9746	f(t) _Total Load
	6000	120	65	6.991	19.937	9411	11826	f(t) _Total Load
	6000	130	81	8.362	23.295	10721	14248	f(t) _Total Load
	6276	131.8	85	8.541	24.065	10965	14829	f(c)_Total Dead Load
10000	7500	90	31	3.580	12.359	5944	6071	Ultimate Moment Strength
	7500	100	39	4.204	13.800	7023	7500	f(t) _Total Load
	7500	110	50	5.145	16.169	8178	9406	f(t) _Total Load
	7500	120	63	6.185	18.849	9411	11514	f(t) _Total Load
	7500	130	79	7.444	22.184	10721	13952	f(t) _Total Load
	7500	138.3	99	8.598	25.653	11864	16336	f(t) _Total Load
12000	9000	90	31	3.359	12.187	5944	6071	Ultimate Moment Strength
	9000	100	38	3.818	13.210	7393	7323	f(t) _Total Load
	9000	110	49	4.688	15.547	8178	9236	f(t) _Total Load
	9000	120	61	5.557	17.873	9411	11200	f(t) _Total Load
	9000	130	77	6.731	21.196	10721	13655	f(t) _Total Load
	9000	139.5	99	7.944	25.035	12043	16344	f(t) _Total Load

Table C.13. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 11.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	36	5.110	14.736	6447	7035	f(t) Total Load
	4500	100	47	6.337	17.363	7611	9006	f(t) _Total Load
	4558	110	60	7.665	20.301	8857	11222	f(t) _Total Load
	4764	112.2	63	7.813	20.826	9146	11712	f(c)_Total Dead Load
8000	6000	90	34	4.301	13.526	6447	6655	f(t) Total Load
	6000	100	45	5.393	16.094	7611	8624	f(t) _Total Load
	6000	110	57	6.510	18.692	8857	10684	f(t) _Total Load
	6000	120	73	7.940	22.200	10185	13245	f(t) _Total Load
	6448	127.6	87	8.847	24.938	11245	15335	f(c)_Total Dead Load
10000	7500	90	33	3.836	12.869	6447	6473	Ultimate Moment Strength/f(t) T L
	7500	100	43	4.704	15.001	7611	8270	f(t) _Total Load
	7500	110	56	5.854	17.958	8857	10520	f(t) _Total Load
	7500	120	71	7.053	21.087	10185	12931	f(t) _Total Load
	7500	130	90	8.452	24.883	11598	15716	f(t) _Total Load
	7500	132.7	99	8.840	26.212	11997	16686	f(t) _Total Load
12000	9000	90	33	3.594	12.681	6447	6473	Ultimate Moment Strength
	9000	100	42	4.279	14.390	7611	8093	f(t) _Total Load
	9000	110	54	5.248	16.981	8857	10191	f(t) _Total Load
	9000	120	69	6.365	20.095	10185	12616	f(t) _Total Load
	9000	130	87	7.645	23.738	11598	15335	f(t) _Total Load
	9000	133.9	99	8.174	25.603	12169	16686	f(c)_Total Dead Load

Table C.14. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 14 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	50	7.274	19.883	8682	9640	f(t) Total Load
	5189	100	66	8.514	23.452	10326	12379	f(t) _Total Load
	5973	106.3	77	9.052	25.601	11418	14188	f(t) _Total Load *
8000	6000	90	48	6.220	18.554	8682	9254	f(t) Total Load
	6000	100	63	7.679	22.104	10326	11833	f(t) _Total Load
	6454	110	84	9.356	26.891	12089	15219	f(t) _Total Load
	7228	115.0	99	9.692	28.934	13022	17071	f(t) _Total Load
10000	7500	90	46	5.455	17.422	8682	8895	Ultimate Moment Strength
	7500	100	61	6.809	20.979	10326	11500	f(t) _Total Load
	7500	110	81	8.527	25.650	12089	14763	f(t) _Total Load
	7500	115.9	99	9.518	28.693	13177	17071	f(t) _Total Load
12000	9000	90	45	4.972	16.777	8682	8715	Ultimate Moment Strength/f(t) T L
	9000	100	60	6.234	20.315	10326	11333	f(t) _Total Load
	9000	110	79	7.733	24.628	12089	14444	f(t) _Total Load
	9000	116.9	99	8.808	28.089	13374	17071	f(t) _Total Load

Table C.15. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 16.6 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4699	90	59	8.397	22.616	10493	11311	f(t) Total Load
	5994	98.9	76	9.235	26.098	11622	14199	f(t) Total Load *
8000	6000	90	57	7.374	21.380	10493	10933	f(t) Total Load
	6000	100	77	9.257	26.172	12467	14311	f(t) Total Load
	7347	107.6	99	9.901	29.636	13348	17373	f(t) Total Load
10000	7500	90	55	6.527	20.256	10493	10588	f(t) Total Load
	7500	100	75	8.291	25.033	12467	13979	f(t) Total Load
	7500	108.3	99	9.793	29.447	14213	17373	f(t) Total Load
12000	9000	90	53	5.862	19.232	10493	10235	f(t) Total Load
	9000	100	72	7.415	23.684	12467	13479	f(t) Total Load
	9000	109.3	99	9.065	28.838	14428	17373	f(t) Total Load

Table C.16. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	21	4.136	12.758	5428	5840	f(t)_Total Load
	4500	100	27	5.007	14.590	6420	7347	f(t)_Total Load
	4500	110	34	6.021	16.750	7483	9048	f(t)_Total Load
	4500	120	42	7.251	19.371	8617	10976	f(t)_Total Load
		120.9	43	7.371	19.700	8727	11205	f(c)_Total Dead Load
8000	6000	90	20	3.510	11.818	5428	5560	Ultimate Moment Strength/f(t)_T L
	6000	100	25	4.146	13.161	6420	6865	f(t)_Total Load
	6000	110	32	5.078	15.341	7483	8588	f(t)_Total Load
	6000	120	40	6.102	17.757	8618	10459	f(t)_Total Load
	6000	130	50	7.415	20.955	9824	12704	f(t)_Total Load
	6301	138.3	59	8.336	23.486	10884	14549	f(c)_Total Dead Load
10000	7500	90	20	3.250	11.612	5428	5560	Ultimate Moment Strength
	7500	100	24	3.630	12.318	6420	6606	Ultimate Moment Strength/f(t)_T L
	7500	110	31	4.490	14.487	7483	8348	f(t)_Total Load
	7500	120	39	5.425	16.870	8618	10229	f(t)_Total Load
	7500	130	48	6.461	19.515	9824	12250	f(t)_Total Load
	7500	140	59	7.695	22.767	11104	14558	f(t)_Total Load
	7774	150	73	9.089	26.848	12457	17172	f(t)_Total Load
	8195	152.9	77	9.302	27.846	12863	17874	f(c)_Total Dead Load
12000	9000	90	20	3.056	11.459	5428	5560	Ultimate Moment Strength
	9000	100	24	3.406	12.145	6420	6606	Ultimate Moment Strength/f(t)_T L
	9000	110	30	4.034	13.723	7483	8107	f(t)_Total Load
	9000	120	38	4.901	16.083	8618	9998	f(t)_Total Load
	9000	130	47	5.864	18.707	9824	12030	f(t)_Total Load
	9000	140	58	7.024	21.967	11104	14366	f(t)_Total Load
	9000	150	71	8.312	25.665	12457	16843	f(t)_Total Load
	9371	160	89	9.875	30.770	13883	19832	f(t)_Total Load
	9937	162.8	97	10.121	32.381	14292	20652	Ultimate Moment Strength/f(t)_T L

Table C.17. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 10 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	23	4.632	13.738	6258	6418	f(t) _Total Load
	4500	100	30	5.773	16.172	7023	8227	f(t) _Total Load
	4500	110	38	6.978	18.755	8178	10187	f(t) _Total Load
	4732	117.1	44	7.691	20.487	9040	11613	f(c)_Total Dead Load
8000	6000	90	22	3.946	12.761	6258	6136	Ultimate Moment Strength/f(t)_T L
	6000	100	28	4.773	14.587	7023	7702	f(t) _Total Load
	6000	110	36	5.852	17.135	8178	9663	f(t) _Total Load
	6000	120	45	7.037	19.931	9411	11785	f(t) _Total Load
	6026	130	56	8.455	23.387	10721	14244	f(t) _Total Load
	6557	133.8	61	8.764	24.719	11242	15303	f(c)_Total Dead Load
10000	7500	90	22	3.643	12.525	6258	6136	Ultimate Moment Strength
	7500	100	27	4.211	13.739	7023	7452	f(t) _Total Load
	7500	110	35	5.195	16.248	8178	9421	f(t) _Total Load
	7500	120	44	6.279	19.009	9411	11554	f(t) _Total Load
	7500	130	54	7.452	21.992	10721	13822	f(t) _Total Load
	7500	140	68	9.021	26.255	12109	16708	f(t) _Total Load
	8556	147.9	80	9.795	29.473	13261	18975	f(c)_Total Dead Load
12000	9000	90	22	3.417	12.348	6258	6136	Ultimate Moment Strength
	9000	100	26	3.763	12.952	7023	7191	Ultimate Moment Strength/f(t)_T L
	9000	110	34	4.686	15.461	8178	9179	f(t) _Total Load
	9000	120	43	5.692	18.197	9411	11321	f(t) _Total Load
	9000	130	53	6.784	21.161	10721	13600	f(t) _Total Load
	9000	140	66	8.123	24.984	12109	16317	f(t) _Total Load
	9000	150	82	9.748	29.713	13577	19362	f(t) _Total Load
	10093	155.9	99	10.468	33.556	14477	21626	f(t) _Total Load

Table C.18. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 11.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	25	5.122	14.705	6447	6994	f(t) _Total Load
	4500	100	33	6.434	17.538	7611	9048	f(t) _Total Load
	4603	110	42	7.756	20.486	8857	11265	f(t) _Total Load
	4909	113.6	45	7.975	21.223	9319	11988	f(c)_Total Dead Load
8000	6000	90	24	4.378	13.692	6447	6710	Ultimate Moment Strength/f(t)_T L
	6000	100	31	5.363	15.925	7611	8518	f(t) _Total Load
	6000	110	40	6.610	18.882	8857	10735	f(t) _Total Load
	6000	120	50	7.951	22.042	10185	13108	f(t) _Total Load
	6699	129.6	62	9.059	25.485	11544	15758	f(c)_Total Dead Load
10000	7500	90	24	4.033	13.426	6786	6710	Ultimate Moment Strength
	7500	100	30	4.747	15.051	7611	8267	f(t) _Total Load
	7500	110	39	5.886	17.965	8857	10493	f(t) _Total Load
	7500	120	49	7.113	21.090	10185	12875	f(t) _Total Load
	7500	130	61	8.497	24.698	11598	15545	f(t) _Total Load
	8281	140	77	9.902	29.375	13093	18857	f(t) _Total Load
	8902	143.2	83	10.229	31.004	13592	19996	f(c)_Total Dead Load
12000	9000	90	24	3.775	13.228	6786	6710	Ultimate Moment Strength
	9000	100	29	4.270	14.272	7611	8014	f(t) _Total Load
	9000	110	38	5.326	17.157	8857	10249	f(t) _Total Load
	9000	120	48	6.466	20.257	10185	12641	f(t) _Total Load
	9000	130	60	7.767	23.874	11598	15331	f(t) _Total Load
	9000	140	75	9.319	28.326	13093	18466	f(t) _Total Load
	10135	149.5	99	10.715	34.224	14595	22268	f(t) _Total Load

Table C.19. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 14 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	35	7.358	20.031	8682	9652	f(t) _Total Load
	5233	100	46	8.637	23.710	10326	12405	f(t) _Total Load
	5977	106.3	53	9.163	25.733	11418	14110	f(t) _Total Load *
8000	6000	90	33	6.182	18.370	8682	9116	f(t) _Total Load
	6000	100	44	7.812	22.359	10326	11868	f(t) _Total Load
	6487	110	58	9.495	27.129	12089	15193	f(t) _Total Load
	7902	118.6	72	10.368	31.105	13701	18272	f(t) _Total Load *
10000	7500	90	32	5.491	17.454	8682	8861	Ultimate Moment Strength/f(t)_T.L.
	7500	100	43	6.979	21.390	10326	11621	f(t) _Total Load
	7500	110	56	8.665	25.876	12089	14741	f(t) _Total Load
	8099	120	74	10.478	31.614	13969	18700	f(t) _Total Load
	9976	127.9	94	11.364	36.719	15529	22469	f(t) _Total Load *
12000	9000	90	32	5.117	17.180	8682	8861	Ultimate Moment Strength
	9000	100	42	6.337	20.544	10326	11374	f(t) _Total Load
	9000	110	54	7.781	24.574	12089	14286	f(t) _Total Load
	9000	120	72	9.785	30.507	13969	18272	f(t) _Total Load
	10277	129.5	99	11.410	37.373	15866	23106	f(t) _Total Load

Table C.20. U54 Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 16.6 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4703	90.0	41	8.5	22.7	9968	11268	f(t) _Total Load
	5932	95.8	48	8.8	24.7	11044	13037	f(t) _Total Load **
8000	6000	90.0	40	7.5	21.6	9968	10982	f(t) _Total Load
	7759	100.0	53	8.5	25.8	11844	14227	f(t) _Total Load
	7965	101.9	55	8.6	26.3	12215	14706	f(t) _Total Load **
10000	7500	90.0	38	6.5	20.1	9968	10473	f(t) _Total Load
	7500	100.0	52	8.5	25.3	11844	13961	f(t) _Total Load
	9808	110.0	68	9.3	29.9	13852	17696	f(t) _Total Load
	9777	110.1	68	9.3	29.9	13863	17696	f(t) _Total Load **
12000	9000	90.0	37	5.9	19.3	9968	10217	f(t) _Total Load
	9000	100.0	50	7.6	24.0	11844	13488	f(t) _Total Load
	9350	110.0	67	9.4	29.6	13852	17470	f(t) _Total Load
	10105	120.0	95	11.6	37.7	15993	23141	f(t) _Total Load
	10343	120.9	99	11.7	38.3	16198	23722	f(t) _Total Load

Table C.21. Controlling Limit States and Maximum Span Lengths for $f_t = 6\sqrt{f'_c}$ and $f_t = 7.5\sqrt{f'_c}$ (U54 Beams - AASHTO LRFD Specifications – Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	$f_t = 6\sqrt{f'_c}$				$f_t = 7.5\sqrt{f'_c}$				% Diff. Max. Span	Diff. No. Strands
		f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State	f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State		
6000	8.5	4500	120.9	43	f (c) _T D L	4500	120.3	40	f (c) _T D L	-0.6	-3
	10.0	4732	117.1	44	f (c) _T D L	4500	116.4	41	f (c) _T D L	-0.6	-3
	11.5	4909	113.6	45	f (c) _T D L	4558	112.9	42	f (c) _T D L	-0.6	-3
	14.0	5977	106.3	53	f(t) _T L *	5947	108.3	53	f(t) _T L *	1.9	0
	16.6	5932	95.8	48	f (t) _T L **	5895	98.5	49	f (t) _T L **	2.8	1
8000	8.5	6301	138.3	59	f (c) _T D L	6000	137.9	55	f (c) _T D L	-0.3	-4
	10.0	6557	133.8	61	f (c) _T D L	6113	133.5	57	f (c) _T D L	-0.3	-4
	11.5	6699	129.6	62	f (c) _T D L	6367	129.4	59	f (c) _T D L	-0.2	-3
	14.0	7902	118.6	72	f(t) _T L *	7978	121.4	73	f(t) _T L *	2.3	1
	16.6	7965	101.9	55	f (t) _T L **	7964	107.5	61	f (t) _T L **	5.5	6
10000	8.5	8195	152.9	77	f (c) _T D L	7647	152.6	72	f (c) _T D L	-0.2	-5
	10.0	8556	147.9	80	f (c) _T D L	8010	147.6	75	f (c) _T D L	-0.2	-5
	11.5	8902	143.2	83	f (c) _T D L	8260	143.0	77	f (c) _T D L	-0.2	-6
	14.0	9976	127.9	94	f(t) _T L *	9958	130.3	94	f(t) _T L *	1.9	0
	16.6	9777	110.1	68	f (t) _T L **	9815	115.1	74	f (t) _T L **	4.6	6
12000	8.5	9937	162.8	97	f (t)_T L / f (c) _T D L	9510	164.5	91	f (t)_T L / f (c) _TDL	1.0	-6
	10.0	10093	155.9	99	f (t) _T L[99]	9771	158.0	94	f (t)_T L / f (c) _TDL	1.3	-5
	11.5	10135	149.5	99	f (t) _T L[99]	10004	152.2	97	f (t)_T L / f (c) _TDL	1.8	-2
	14.0	10277	129.5	99	f (t) _T L[99]	10258	132.2	99	f (t) _T L[99]	2.0	0
	16.6	10343	120.9	99	f (t) _T L[99]	10324	123.4	99	f (t) _T L[99]	2.1	0

Table C.22. Controlling Limit States and Maximum Span Lengths for $f_t = 6\sqrt{f'_c}$ and $f_t = 8\sqrt{f'_c}$ (U54 Beams - AASHTO LRFD Specifications – Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	$f_t = 6\sqrt{f'_c}$				$f_t = 8\sqrt{f'_c}$				% Diff. Max. Span	Diff. No. Strands
		f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State	f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State		
6000	8.5	4500	120.9	43	f (c) _T D L	4500	120.0	39	f (c) _T D L	-0.7	-4
	10.0	4732	117.1	44	f (c) _T D L	4500	116.2	40	f (c) _T D L	-0.7	-4
	11.5	4909	113.6	45	f (c) _T D L	4500	112.7	41	f (c) _T D L	-0.7	-4
	14.0	5977	106.3	53	f(t) _T L *	5936	109.0	53	f(t) _T L *	2.6	0
	16.6	5932	95.8	48	f (t) _T L **	5770	99.1	49	f (t) _T L **	3.4	1
8000	8.5	6301	138.3	59	f (c) _T D L	6000	137.9	54	f (c) _T D L	-0.3	-5
	10.0	6557	133.8	61	f (c) _T D L	6000	133.4	56	f (c) _T D L	-0.3	-5
	11.5	6699	129.6	62	f (c) _T D L	6145	129.2	57	f (c) _T D L	-0.3	-5
	14.0	7902	118.6	72	f(t) _T L *	7973	122.1	73	f(t) _T L *	3.0	1
	16.6	7965	101.9	55	f (t) _T L **	7821	108.7	62	f (t) _T L **	6.7	7
10000	8.5	8195	152.9	77	f (c) _T D L	7532	152.4	70	f (c) _T D L	-0.3	-7
	10.0	8556	147.9	80	f (c) _T D L	7791	147.5	73	f (c) _T D L	-0.3	-7
	11.5	8902	143.2	83	f (c) _T D L	8041	142.9	75	f (c) _T D L	-0.2	-8
	14.0	9976	127.9	94	f(t) _T L *	9952	131.1	94	f(t) _T L *	2.6	0
	16.6	9777	110.1	68	f (t) _T L **	9432	115.8	74	f (t) _T L **	5.3	6
12000	8.5	9937	162.8	97	f (t)_T L / f (c) _TDL	9426	164.7	90	f (c) _T D L	1.2	-7
	10.0	10093	155.9	99	f (t) _T L[99]	9769	158.2	94	f (c) _T D L	1.5	-5
	11.5	10135	149.5	99	f (t) _T L[99]	9873	152.8	95	f (t)_T L / f (c) _TDL	2.2	-4
	14.0	10277	129.5	99	f (t) _T L[99]	10252	133.0	99	f (t) _T L[99]	2.7	0
	16.6	10343	120.9	99	f (t) _T L[99]	10318	124.2	99	f (t) _T L[99]	2.7	0

APPENDIX D

RESULTS FOR TYPE IV BEAMS

Note: For a description of the controlling limit states, refer to Table 7.2.

Table D.1. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 4.25 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	18	3.795	10.751	3171	3643	Ultimate Moment Strength/f(t)_T L
	4500	100	22	4.269	11.682	3762	4158	f(t) Total Load
	4500	110	28	5.114	13.655	4396	5137	f(t) Total Load
	4500	120	36	6.294	16.559	5072	6368	f(t) Total Load
	4826	126.5	42	6.698	18.406	5536	7188	f(c) Total Dead Load
8000	6000	90	18	3.448	10.452	3171	3451	Ultimate Moment Strength
	6000	100	22	3.863	11.340	3762	4140	f(t) Total Load
	6000	110	28	4.604	13.241	4396	5109	f(t) Total Load
	6000	120	34	5.249	14.814	5072	6020	f(t) Total Load
	6000	130	44	6.491	18.294	5791	7396	f(t) Total Load
	6053	140	54	7.490	21.092	6553	8571	f(t) Total Load
	6005	141.8	56	7.691	21.605	6698	8775	f(c) Total Dead Load
10000	7500	90	18	3.208	10.245	3171	3451	Ultimate Moment Strength
	7500	100	22	3.583	11.103	3762	4140	f(t) Total Load
	7500	110	26	3.883	11.682	4396	4796	f(t) Total Load
	7500	120	34	4.833	14.487	5072	6020	f(t) Total Load
	7500	130	42	5.638	16.793	5791	7139	f(t) Total Load
	7500	140	52	6.603	19.674	6553	8369	f(t) Total Load
	7500	150	66	7.805	23.525	7358	9744	f(t) Total Load
	7500	152.6	70	8.035	24.320	7573	10021	f(c) Total Dead Load
12000	9000	90	18	3.030	9.090	3171	3451	Ultimate Moment Strength
	9000	100	20	3.005	9.570	3762	3798	Ultimate Moment Strength/f(t)_T L
	9000	110	26	3.649	11.489	4396	4796	f(t) Total Load
	9000	120	32	4.194	13.029	5072	5721	f(t) Total Load
	9000	130	40	4.967	15.407	5791	6873	f(t) Total Load
	9000	140	50	5.887	18.358	6553	8148	f(t) Total Load
	9000	150	64	7.064	22.388	7358	9621	f(t) Total Load
	9000	155.5	76	7.721	25.011	7819	10385	f(t) Total Load [e]

Table D.2. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	20	4.321	11.981	3514	3851	Ultimate Moment Strength/f(t)_T L
	4500	100	26	5.254	14.172	4159	4884	f(t) Total Load
	4500	110	32	6.036	15.915	4849	5851	f(t) Total Load
	4542	120	40	7.091	18.480	5583	7058	f(t) Total Load
	5077	123.4	44	7.340	19.729	5843	7635	f(c) Total Dead Load
8000	6000	90	20	3.906	11.629	3514	3839	Ultimate Moment Strength
	6000	100	24	4.315	12.436	4159	4535	f(t) Total Load
	6000	110	32	5.411	15.421	4849	5820	f(t) Total Load
	6000	120	38	6.010	16.794	5583	6727	f(t) Total Load
	6000	130	48	7.166	19.964	6362	8106	f(t) Total Load
	6214	137.9	58	8.098	22.871	7011	9275	f(c) Total Dead Load
10000	7500	90	20	3.619	11.385	3514	3839	Ultimate Moment Strength
	7500	100	24	3.989	12.165	4159	4535	f(t) Total Load
	7500	110	30	4.619	13.845	4849	5505	f(t) Total Load
	7500	120	38	5.519	16.419	5583	6727	f(t) Total Load
	7500	130	48	6.566	19.530	6362	8109	f(t) Total Load
	7500	140	60	7.661	22.880	7186	9513	f(t) Total Load
	7500	146.7	72	8.483	25.699	7767	10522	f(t) Total Load / f(c)_T D L
12000	9000	90	20	3.405	11.203	3514	3839	Ultimate Moment Strength
	9000	100	22	3.381	10.623	4159	4189	Ultimate Moment Strength/f(t)_T L
	9000	110	30	4.324	13.607	4849	5505	f(t) Total Load
	9000	120	36	4.854	15.020	5583	6438	f(t) Total Load
	9000	130	46	5.851	18.203	6362	7855	f(t) Total Load
	9000	140	58	6.892	21.624	7186	9310	f(t) Total Load
	9000	148.9	76	8.014	25.885	7956	10784	f(t) Total Load [e]

Table D.3. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 5.75 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	22	4.839	13.187	3857	4239	f(t) _ Total Load
	4500	100	28	5.722	15.192	4556	5265	f(t) _ Total Load
	4500	110	36	6.932	18.099	5301	6565	f(t) _ Total Load
	5065	120	44	7.556	20.113	6094	7774	f(t) _ Total Load
	5328	120.5	46	7.740	20.996	6133	8043	f(c)_Total Dead Load
8000	6000	90	20	3.912	11.380	3857	3869	Ultimate Moment Strength/f(t)_T L
	6000	100	28	5.132	14.717	4556	5244	f(t) _ Total Load
	6000	110	34	5.811	16.309	5301	6215	f(t) _ Total Load
	6000	120	44	7.068	19.751	6094	7705	f(t) _ Total Load
	6000	130	56	8.380	23.426	6933	9284	f(t) _ Total Load
	6388	134.0	62	8.735	24.966	7283	9954	f(c)_Total Dead Load
10000	7500	90	20	3.625	11.137	3857	3869	Ultimate Moment Strength/f(t)_T L
	7500	100	26	4.362	13.135	4556	4913	f(t) _ Total Load
	7500	110	34	5.337	15.939	5301	6215	f(t) _ Total Load
	7500	120	42	6.163	18.226	6094	7417	f(t) _ Total Load
	7500	130	54	7.415	22.031	6933	9051	f(t) _ Total Load
	7500	140	70	8.666	26.113	7819	10705	f(t) _ Total Load
	7500	141.7	76	8.953	27.311	7977	11079	f(t) _ Total Load / f(c)_T D L
12000	9000	90	20	3.412	10.955	3857	3869	Ultimate Moment Strength/f(t)_T L
	9000	100	26	4.087	12.909	4556	4913	f(t) _ Total Load
	9000	110	32	4.659	14.464	5301	5895	f(t) _ Total Load
	9000	120	42	5.742	17.912	6094	7417	f(t) _ Total Load
	9000	130	52	6.641	20.703	6933	8795	f(t) _ Total Load
	9000	140	68	7.904	25.083	7819	10567	f(t) _ Total Load
	9000	142.9	76	8.266	26.649	8087	11117	f(t) _ Total Load [e]

Table D.4. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 7 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	26	5.814	15.531	4429	4989	f(t) _ Total Load
	4500	100	34	7.073	18.538	5217	6342	f(t) _ Total Load
	4772	110	42	7.944	20.821	6056	7606	f(t) _ Total Load
	5236	115.7	48	8.400	22.519	6552	8500	f(c)_Total Dead Load
8000	6000	90	24	4.809	13.759	4429	4630	Ultimate Moment Strength/f(t)_T L
	6000	100	32	5.931	16.725	5217	5985	f(t) _ Total Load
	6000	110	40	6.910	19.203	6056	7263	f(t) _ Total Load
	6000	120	52	8.342	23.166	6944	9007	f(t) _ Total Load
	6556	128.0	66	9.393	27.083	7692	10733	f(c)_Total Dead Load
10000	7500	90	24	4.430	13.447	4429	4630	Ultimate Moment Strength/f(t)_T L
	7500	100	30	5.091	15.123	5217	5651	f(t) _ Total Load
	7500	110	40	6.329	18.766	6056	7263	f(t) _ Total Load
	7500	120	50	7.353	21.964	6944	8730	f(t) _ Total Load
	7500	130	66	8.806	26.304	7884	10737	f(t) _ Total Load
	7500	133.1	76	9.329	28.392	8181	11548	f(t) _ Total Load [e]
12000	9000	90	24	4.148	13.214	4429	4630	Ultimate Moment Strength
	9000	100	30	4.755	14.855	5217	5651	f(t) _ Total Load
	9000	110	38	5.605	17.355	6056	6955	f(t) _ Total Load
	9000	120	50	6.837	21.326	6944	8730	f(t) _ Total Load
	9000	130	64	7.999	25.164	7884	10525	f(t) _ Total Load
	9000	134.2	76	8.616	27.722	8295	11563	f(t) _ Total Load [e]

Table D.5. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	30	6.724	17.626	5115	5733	f(t) _ Total Load
	4500	100	40	8.302	21.468	6011	7406	f(t) _ Total Load
	5702	110	52	9.024	24.857	6961	9242	f(t) _ Total Load
	5704	110.6	52	8.993	24.714	7023	9242	f(c)_Total Dead Load
8000	6000	90	30	6.007	17.058	5115	5716	f(t) _ Total Load
	6000	100	38	7.057	19.692	6011	7058	f(t) _ Total Load
	6000	110	50	8.544	23.758	6961	8904	f(t) _ Total Load
	6528	120	66	9.787	28.099	7966	11017	f(t) _ Total Load
	6565	121.2	70	10.035	29.047	8087	11433	f(c)_Total Dead Load
10000	7500	90	28	5.157	15.444	5115	5371	Ultimate Moment Strength/f(t)_T L
	7500	100	36	6.148	18.144	6011	6737	f(t) _ Total Load
	7500	110	48	7.537	22.273	6961	8612	f(t) _ Total Load
	7500	120	64	9.062	27.035	7966	10785	f(t) _ Total Load
	7500	124.2	76	9.687	29.444	8408	11955	f(t) _ Total Load [e]
12000	9000	90	28	4.814	15.168	5115	5371	Ultimate Moment Strength/f(t)_T L
	9000	100	36	5.725	17.819	6011	6737	f(t) _ Total Load
	9000	110	46	6.753	20.927	6961	8317	f(t) _ Total Load
	9000	120	62	8.216	25.830	7966	10538	f(t) _ Total Load
	9000	125.6	78	9.001	29.078	8556	12091	f(t) _ Total Load [e]

Table D.6. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.5 in., Girder Spacing = 9 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	32	7.168	18.729	5344	6097	f(t) _ Total Load
	4742	100	42	8.509	22.256	6276	7756	f(t) _ Total Load
	5941	109.1	54	9.215	25.684	7175	9583	f(c)_Total Dead Load *
8000	6000	90	30	6.011	16.897	5344	5733	f(t) _ Total Load
	6000	100	40	7.403	20.600	6276	7406	f(t) _ Total Load
	6000	110	54	9.138	25.459	7263	9534	f(t) _ Total Load
	6645	119.1	70	10.076	29.222	8212	11519	f(t) _ Total Load / f(c)_T D L
10000	7500	90	30	5.512	16.501	5344	5733	f(t) _ Total Load
	7500	100	40	6.771	20.127	6276	7406	f(t) _ Total Load
	7500	110	52	8.080	23.966	7263	9243	f(t) _ Total Load
	7500	120	70	9.554	28.714	8306	11519	f(t) _ Total Load
	7500	121.9	78	9.842	30.048	8511	12198	f(t) _ Total Load [e]
12000	9000	90	28	4.818	15.006	5344	5386	Ultimate Moment Strength/f(t)_T L
	9000	100	38	6.015	18.705	6276	7085	f(t) _ Total Load
	9000	110	50	7.257	22.619	7263	8950	f(t) _ Total Load
	9000	120	68	8.737	27.695	8306	11315	f(t) _ Total Load
	9000	123.0	78	9.093	29.371	8630	12199	f(t) _ Total Load [e]

Table D.7. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 4.25 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	12	3.623	10.214	3171	3324	Ultimate Moment Strength/f(t)_T L
	4500	100	16	4.532	12.382	3762	4311	f(t) Total Load
	4500	110	20	5.353	14.255	4396	5259	f(t) Total Load
	4500	120	24	6.092	15.850	5072	6169	f(t) Total Load
	4906	128.8	30	7.133	18.905	5701	7408	f(c)_ Total Dead Load
8000	6000	90	12	3.299	9.933	3171	3313	Ultimate Moment Strength/f(t)_T L
	6000	100	16	4.093	12.018	3762	4292	f(t) Total Load
	6000	110	20	4.813	13.821	4396	5230	f(t) Total Load
	6000	120	24	5.465	15.364	5072	6129	f(t) Total Load
	6000	130	30	6.523	18.205	5791	7355	f(t) Total Load
	6131	140	36	7.433	20.690	6553	8480	f(t) Total Load
	6385	145.5	40	7.909	22.249	6992	9142	f(c) Total Dead Load
10000	7500	90	12	3.075	9.739	3171	3313	Ultimate Moment Strength/f(t)_T L
	7500	100	14	3.213	9.784	3762	3808	f(t) Total Load
	7500	110	18	3.877	11.595	4396	4766	f(t) Total Load
	7500	120	24	5.028	15.025	5072	6129	f(t) Total Load
	7500	130	28	5.492	16.105	5791	6961	f(t) Total Load
	7500	140	36	6.870	20.297	6553	8499	f(t) Total Load
	7500	150	42	7.592	22.248	7358	9504	f(t) Total Load
	8110	159.2	52	8.763	26.629	8135	10961	f(c) Total Dead Load
12000	9000	90	12	2.909	9.594	3171	3313	Ultimate Moment Strength/f(t)_T L
	9000	100	14	3.036	9.631	3762	3808	f(t) Total Load
	9000	110	18	3.645	11.403	4396	4766	f(t) Total Load
	9000	120	22	4.198	12.922	5072	5684	f(t) Total Load
	9000	130	28	5.129	15.831	5791	6962	f(t) Total Load
	9000	140	34	5.959	18.343	6553	8137	f(t) Total Load
	9000	150	42	7.066	21.900	7358	9533	f(t) Total Load
	9000	160	52	8.362	26.197	8205	11032	f(t) Total Load
	9251	169.1	66	9.860	31.844	9016	12624	f(c) Total Dead Load

Table D.8. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	14	4.376	12.090	3514	3860	Ultimate Moment Strength/f(t)_T L
	4500	100	18	5.262	14.129	4159	4851	f(t) Total Load
	4500	110	22	6.063	15.878	4849	5809	f(t) Total Load
	4500	120	28	7.364	19.037	5583	7146	f(t) Total Load
	5289	125.9	32	7.686	20.705	6038	7993	f(c) Total Dead Load
8000	6000	90	14	3.954	11.733	3514	3848	Ultimate Moment Strength/f(t)_T L
	6000	100	18	4.730	13.695	4159	4831	f(t) Total Load
	6000	110	22	5.436	15.384	4849	5779	f(t) Total Load
	6000	120	26	6.034	16.711	5583	6674	f(t) Total Load
	6000	130	34	7.600	21.091	6362	8335	f(t) Total Load
	6724	140	42	8.535	24.479	7186	9775	f(t) Total Load
	6732	141.8	42	8.425	24.006	7342	9775	f(c) Total Dead Load
10000	7500	90	14	3.663	11.485	3514	3848	Ultimate Moment Strength/f(t)_T L
	7500	100	16	3.796	11.454	4159	4344	Ultimate Moment Strength/f(t)_T L
	7500	110	22	4.999	15.040	4849	5779	f(t) Total Load
	7500	120	26	5.542	16.336	5583	6674	f(t) Total Load
	7500	130	32	6.482	18.989	6362	7937	f(t) Total Load
	7500	140	40	7.749	22.759	7186	9452	f(t) Total Load
	7725	150	50	9.096	27.159	8055	11077	f(t) Total Load
	8340	154.6	56	9.601	29.584	8473	11909	f(c) Total Dead Load
12000	9000	90	14	3.445	11.301	3514	3848	Ultimate Moment Strength
	9000	100	16	3.569	11.264	4159	4344	Ultimate Moment Strength/f(t)_T L
	9000	110	20	4.167	12.922	4849	5309	f(t) _ Total Load
	9000	120	26	5.172	16.054	5583	6674	f(t) _ Total Load
	9000	130	32	6.039	18.669	6362	7938	f(t) _ Total Load
	9000	140	40	7.208	22.396	7186	9464	f(t) _ Total Load
	9000	150	48	8.208	25.511	8055	10814	f(t) _ Total Load
	9000	160	62	9.924	31.456	8970	12723	f(t) _ Total Load
	9254	163.0	70	10.580	34.381	9256	13487	f(c)_Total Dead Load

Table D.9. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 5.75 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	16	5.111	13.912	3857	4397	f(t) Total Load
	4500	100	20	5.977	15.831	4556	5390	f(t) Total Load
	4500	110	24	6.761	17.464	5301	6355	f(t) Total Load
	5268	120	32	8.066	21.717	6094	8122	f(t) Total Load
	5671	123.2	34	8.185	22.434	6359	8559	f(c) Total Dead Load*
8000	6000	90	14	3.961	11.479	3857	3878	Ultimate Moment Strength/f(t)_T L
	6000	100	20	5.356	15.335	4556	5369	f(t) Total Load
	6000	110	24	6.049	16.915	5301	6324	f(t) Total Load
	6000	120	30	7.131	19.739	6094	7644	f(t) Total Load
	6000	130	38	8.604	23.769	6933	9294	f(t) Total Load
	7444	138.6	46	9.115	26.953	7692	10730	f(c) Total Dead Load
10000	7500	90	14	3.669	11.231	3857	3878	Ultimate Moment Strength/f(t)_T L
	7500	100	18	4.369	13.085	4556	4879	Ultimate Moment Strength/f(t)_T L
	7500	110	24	5.552	16.532	5301	6324	f(t) Total Load
	7500	120	30	6.532	19.302	6094	7644	f(t) Total Load
	7500	130	36	7.443	21.771	6933	8912	f(t) Total Load
	7500	140	46	9.013	26.572	7819	10753	f(t) Total Load
	8628	150	58	10.025	31.237	8753	12559	f(t) Total Load
	8631	150.4	58	10.004	31.132	8792	12559	f(c) Total Dead Load
12000	9000	90	14	3.452	11.047	3857	3878	Ultimate Moment Strength/f(t)_T L
	9000	100	18	4.094	12.859	4556	4879	Ultimate Moment Strength/f(t)_T L
	9000	110	22	4.681	14.412	5301	5850	f(t) Total Load
	9000	120	28	5.636	17.311	6094	7211	f(t) Total Load
	9000	130	36	6.924	21.411	6933	8913	f(t) Total Load
	9000	140	44	7.995	24.745	7819	10412	f(t) Total Load
	9000	150	56	9.553	29.972	8753	12339	f(t) Total Load
	9375	156.8	74	11.112	36.568	9414	14242	f(c) Total Dead Load

Table D.10. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 7 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	18	5.835	15.522	4429	4952	f(t)_Total load
	4500	100	24	7.365	19.227	5217	6452	f(t) _ Total Load
	4841	110	30	8.370	21.972	6056	7839	f(t) _ Total Load
	5651	117.9	34	8.515	23.046	6751	8734	f(t)_Total Load *
8000	6000	90	18	5.229	15.030	4429	4938	f(t) _ Total Load
	6000	100	22	5.976	16.737	5217	5936	f(t) _ Total Load
	6000	110	28	7.152	19.780	6056	7348	f(t) _ Total Load
	6049	120	36	8.684	23.984	6944	9118	f(t) _ Total Load
	7404	130	46	9.594	28.286	7884	11110	f(t) _ Total Load
	7307	132.9	48	9.869	28.886	8168	11462	f(c)_Total Dead Load
10000	7500	90	16	4.249	12.767	4429	4429	Ultimate Moment Strength/f(t)_T L
	7500	100	22	5.484	16.351	5217	5936	f(t) _ Total Load
	7500	110	28	6.548	19.332	6056	7348	f(t) _ Total Load
	7500	120	34	7.500	21.880	6944	8684	f(t) _ Total Load
	7500	130	44	9.138	26.833	7884	10731	f(t) _ Total Load
	8579	140	58	10.542	32.824	8874	13101	f(t) _ Total Load
	9140	143.4	64	10.930	35.010	9223	13924	f(c)_Total Dead Load
12000	9000	90	16	3.982	12.546	4429	4429	Ultimate Moment Strength/f(t)_T L
	9000	100	22	5.115	16.062	5217	5936	f(t) _ Total Load
	9000	110	26	5.648	17.327	6056	6890	f(t) _ Total Load
	9000	120	34	6.973	21.510	6944	8684	f(t) _ Total Load
	9000	130	44	8.487	26.414	7884	10732	f(t) _ Total Load
	9000	140	56	10.044	31.563	8874	12824	f(t) _ Total Load
	9451	147.7	76	11.565	38.238	9671	15080	f(t) _ Total Load [e]

Table D.11. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	22	7.234	18.964	5115	6025	f(t) _ Total Load
	4500	100	28	8.600	22.130	6011	7489	f(t) _ Total Load
	5622	109.8	34	8.979	24.263	6943	8889	f(t) _ Total Load *
8000	6000	90	20	5.853	16.459	5115	5499	Ultimate Moment Strength/f(t)_T L
	6000	100	26	7.130	19.738	6011	6987	f(t) _ Total Load
	6000	110	34	8.737	24.052	6961	8847	f(t) _ Total Load
	7000	120	44	9.909	28.620	7966	10980	f(t) _ Total Load
	7953	126.9	52	10.533	31.784	8693	12544	f(c) _ Total Dead Load *
10000	7500	90	20	5.371	16.074	5115	5499	Ultimate Moment Strength/f(t)_T L
	7500	100	26	6.525	19.283	6011	6987	f(t) _ Total Load
	7500	110	34	7.984	23.525	6961	8847	f(t) _ Total Load
	7500	120	44	9.634	28.441	7966	10980	f(t) _ Total Load
	8531	130	58	11.027	34.330	9025	13587	f(t) _ Total Load
	9320	135.4	68	11.605	37.621	9625	15020	f(t) _ Total Load / f(c)_T D L
12000	9000	90	20	5.010	15.787	5115	5499	Ultimate Moment Strength/f(t)_T L
	9000	100	26	6.072	18.941	6011	6987	f(t) _ Total Load
	9000	110	32	6.987	21.531	6961	8390	f(t) _ Total Load
	9000	120	42	8.569	26.593	7966	10567	f(t) _ Total Load
	9000	130	56	10.500	33.072	9025	13257	f(t) _ Total Load
	9920	137.8	76	11.710	39.339	9895	15805	f(t) _ Total Load [e]

Table D.12. Type IV Beam Designs - AASHTO Standard Specifications (Strand Diameter = 0.6 in., Girder Spacing = 9 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	22	7.238	18.797	5344	6042	f(t) _ Total Load
	4810	100	30	8.955	23.462	6276	7992	f(t) _ Total Load
	6000	109.8	36	9.262	25.492	7238	9391	f(t) _ Total Load *
8000	6000	90	22	6.459	18.190	5344	6025	f(t) _ Total Load
	6000	100	28	7.666	21.236	6276	7489	f(t) _ Total Load
	6012	110	36	9.243	25.423	7263	9347	f(t) _ Total Load
	7253	120	48	10.541	30.935	8306	11854	f(t) _ Total Load
	7941	124.2	52	10.667	32.183	8757	12621	f(t) _ Total Load *
10000	7500	90	20	5.375	15.907	5344	5514	Ultimate Moment Strength/f(t)_T L
	7500	100	28	7.010	20.751	6276	7489	f(t) _ Total Load
	7500	110	36	8.450	24.879	7263	9347	f(t) _ Total Load
	7500	120	46	10.040	29.568	8306	11464	f(t) _ Total Load
	8806	130	62	11.434	36.061	9406	14364	f(t) _ Total Load
	9651	133.1	70	11.691	38.498	9758	15428	f(t) _ Total Load / f(c)_T D L
12000	9000	90	20	5.014	15.619	5344	5514	Ultimate Moment Strength/f(t)_T L
	9000	100	26	6.077	18.738	6276	7013	f(t) _ Total Load
	9000	110	34	7.422	22.889	7263	8890	f(t) _ Total Load
	9000	120	44	8.949	27.725	8306	11050	f(t) _ Total Load
	9000	130	60	11.080	34.999	9406	14057	f(t) _ Total Load
	10069	134.9	76	11.742	39.651	9968	16012	f(t) _ Total Load [e]

Table D.13. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 4.25 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	20	4.391	13.539	3598	3813	Ultimate Moment Strength/f(t)_T L
	4500	100	26	5.366	15.698	4247	4820	f(t) _ Total Load
	4500	110	34	6.652	18.654	4943	6060	f(t) _ Total Load
	4793	120	42	7.544	20.938	5685	7188	f(t) _ Total Load
	5755	130	52	8.183	23.466	6474	8394	f(t) _ Total Load
	5950	132.9	56	8.529	24.597	6711	8813	f(c)_Total Dead Load
8000	6000	90	20	3.973	13.213	3621	3798	Ultimate Moment Strength/f(t)_T L
	6000	100	26	4.831	15.297	4273	4796	f(t) _ Total Load
	6000	110	32	5.551	16.969	4971	5721	f(t) _ Total Load
	6000	120	42	6.923	20.514	5716	7136	f(t) _ Total Load
	6000	130	52	8.053	23.375	6509	8348	f(t) _ Total Load
	6594	140	64	8.865	26.147	7348	9521	f(t) _ Total Load
	6731	144.1	74	9.488	28.287	7707	10123	f(t)_Total Load / f(c)_T D L
10000	7500	90	20	3.683	12.987	3638	3798	Ultimate Moment Strength
	7500	100	24	4.067	13.789	4293	4477	f(t) _ Total Load
	7500	110	32	5.113	16.651	4993	5721	f(t) _ Total Load
	7500	120	40	6.023	19.071	5741	6872	f(t) _ Total Load
	7500	130	50	7.091	22.002	6535	8137	f(t) _ Total Load
	7500	140	64	8.434	25.883	7377	9581	f(t) _ Total Load
	7500	145.3	76	9.157	28.269	7845	10310	f(t) _ Total Load [e]
12000	9000	90	20	3.468	12.818	3653	3798	Ultimate Moment Strength
	9000	100	24	3.822	13.601	4309	4477	Ultimate Moment Strength/f(t)_T L
	9000	110	30	4.433	15.241	5012	5418	f(t) _ Total Load
	9000	120	38	5.311	17.737	5761	6600	f(t) _ Total Load
	9000	130	48	6.335	20.743	6558	7909	f(t) _ Total Load
	9000	140	62	7.636	24.758	7401	9435	f(t) _ Total Load
	9000	146.6	78	8.571	28.075	7981	10456	f(t) _ Total Load [e]

Table D.14. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	22	4.932	14.701	3910	4204	Ultimate Moment Strength/f(t)_T L
	4500	100	28	5.853	16.677	4608	5210	f(t) _ Total Load
	4500	110	36	7.117	19.521	5356	6477	f(t) _ Total Load
	5311	120	46	8.022	22.480	6153	7910	f(t) _ Total Load
	5986	127.7	54	8.521	24.400	6800	8895	f(t) _ Total Load *
8000	6000	90	22	4.448	14.328	3936	4189	Ultimate Moment Strength/f(t)_T L
	6000	100	28	5.259	16.238	4637	5186	f(t) _ Total Load
	6000	110	36	6.378	19.001	5388	6438	f(t) _ Total Load
	6000	120	46	7.653	22.231	6189	7852	f(t) _ Total Load
	6162	130	58	8.871	25.560	7039	9275	f(t) _ Total Load
	6796	139.2	76	9.819	29.229	7864	10643	Ultimate Moment Strength/f(t)_T L
10000	7500	90	22	4.112	14.070	3956	4189	Ultimate Moment Strength
	7500	100	28	4.847	15.932	4660	5186	f(t) _ Total Load
	7500	110	34	5.490	17.478	5414	6131	f(t) _ Total Load
	7500	120	44	6.694	20.780	6216	7581	f(t) _ Total Load
	7500	130	56	7.955	24.302	7069	9075	f(t) _ Total Load
	7500	140	78	9.506	29.247	7972	10806	f(t) _ Total Load
	7500	140.2	78	9.496	29.206	7992	10806	f(t) _ Total Load [e]
12000	9000	90	22	3.862	13.877	3973	4189	Ultimate Moment Strength
	9000	100	26	4.183	14.514	4679	4863	Ultimate Moment Strength/f(t)_T L
	9000	110	34	5.133	17.223	5435	6131	f(t) _ Total Load
	9000	120	44	6.246	20.478	6239	7581	f(t) _ Total Load
	9000	130	54	7.166	23.107	7094	8858	f(t) _ Total Load
	9000	140	74	8.659	28.036	7999	10684	f(t) _ Total Load
	9000	141.1	78	8.804	28.651	8105	10868	f(t) _ Total Load [e]

Table D.15. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 5.75 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	24	5.466	15.841	4215	4594	f(t) _ Total Load
	4500	100	32	6.803	18.869	4962	5922	f(t) _ Total Load
	4500	110	40	7.959	21.398	5761	7167	f(t) _ Total Load
	5717	120	52	8.793	24.816	6612	8857	f(t) _ Total Load
	5920	124.5	56	9.032	25.562	7014	9351	f(t) _ Total Load *
8000	6000	90	24	4.917	15.426	4244	4578	Ultimate Moment Strength/f(t)_T L
	6000	100	30	5.681	17.162	4995	5571	f(t) _ Total Load
	6000	110	40	7.122	20.825	5797	7125	f(t) _ Total Load
	6000	120	50	8.308	23.763	6652	8533	f(t) _ Total Load
	6552	130	64	9.437	27.397	7559	10159	f(t) _ Total Load
	6777	134.4	76	10.061	29.756	7970	11021	f(c)_Total Dead Load
10000	7500	90	24	4.535	15.137	4267	4578	Ultimate Moment Strength
	7500	100	30	5.229	16.831	5020	5571	f(t) _ Total Load
	7500	110	38	6.203	19.366	5826	6830	f(t) _ Total Load
	7500	120	48	7.323	22.374	6683	8257	f(t) _ Total Load
	7500	140	64	8.953	27.102	7593	10179	f(t) _ Total Load
	7500	135.4	78	9.713	29.708	8103	11170	f(t) _ Total Load [e]
12000	9000	90	24	4.251	14.921	4285	4578	Ultimate Moment Strength
	9000	100	30	4.890	16.584	5041	5571	f(t) _ Total Load
	9000	110	38	5.790	19.079	5849	6830	f(t) _ Total Load
	9000	120	48	6.827	22.049	6709	8257	f(t) _ Total Load
	9000	130	62	8.121	25.972	7621	9979	f(t) _ Total Load
	9000	136.2	78	9.907	29.152	8217	11214	f(t) _ Total Load [e]

Table D.16. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 7 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	28	6.431	17.968	4713	5331	f(t) _ Total Load
	4500	100	36	7.728	20.800	5539	6673	f(t) _ Total Load
	5276	110	46	8.627	23.691	6422	8214	f(t) _ Total Load
	5915	117.8	56	9.401	26.366	7153	9603	f(t) _ Total Load *
8000	6000	90	26	5.344	16.260	4746	4973	Ultimate Moment Strength/f(t)_T L
	6000	100	36	6.914	20.231	5577	6644	f(t) _ Total Load
	6000	110	46	8.205	23.409	6464	8169	f(t) _ Total Load
	6000	120	60	9.728	27.542	7407	10058	f(t) _ Total Load
	6983	127.8	80	10.387	30.980	8185	11719	f(c)_T D L - f(t) _ T L
10000	7500	90	26	4.922	15.945	4773	4973	Ultimate Moment Strength/f(t)_T L
	7500	100	34	5.978	18.685	5607	6316	f(t) _ Total Load
	7500	110	44	7.198	21.937	6497	7870	f(t) _ Total Load
	7500	120	58	8.721	26.212	7444	9791	f(t) _ Total Load
	7500	128.2	78	10.019	30.425	8263	11659	f(t) _ Total Load [e]
12000	9000	90	26	4.606	15.709	4795	4973	Ultimate Moment Strength/f(t)_T L
	9000	100	34	5.581	18.403	5632	6316	f(t) _ Total Load
	9000	110	44	6.709	21.610	6525	7870	f(t) _ Total Load
	9000	120	56	7.882	25.011	7474	9537	f(t) _ Total Load
	9000	129.1	78	9.291	29.855	8385	11674	f(t) _ Total Load [e]

Table D.17. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	32	7.367	19.945	5294	6078	f(t) _ Total Load
	4728	100	42	8.786	23.370	6214	7725	f(t) _ Total Load
	5866	110	56	9.838	27.383	7195	9808	f(t) _ Total Load
	5869	111.0	56	9.786	27.168	7297	9808	f(t) _ Total Load *
8000	6000	90	32	6.591	19.388	5333	6059	f(t) _ Total Load
	6000	100	42	8.003	22.841	6258	7693	f(t) _ Total Load
	6000	110	56	9.472	26.489	7244	9481	f(t) _ Total Load
	7006	120	78	10.632	31.451	8292	12085	f(t) _ Total Load
	6990	120.3	78	10.628	31.396	8328	12085	f(t) _ Total Load [e]
10000	7500	90	30	5.679	17.838	5364	5716	f(t) _ Total Load
	7500	100	40	7.009	21.360	6293	7377	f(t) _ Total Load
	7500	110	52	8.392	25.079	7283	9194	f(t) _ Total Load
	7500	120	74	10.193	30.575	8335	11804	f(t) _ Total Load
	7500	120.9	78	10.313	31.116	8434	12089	f(t) _ Total Load [e]
12000	9000	90	30	5.304	17.565	5390	5716	Ultimate Moment Strength/f(t)_T L
	9000	100	40	6.533	21.034	6322	7377	f(t) _ Total Load
	9000	110	52	7.814	24.710	7315	9194	f(t) _ Total Load
	9000	120	72	9.386	29.731	8370	11636	f(t) _ Total Load
	9000	121.7	78	9.566	30.545	9558	12091	f(t) _ Total Load [e]

Table D.18. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.5 in., Girder Spacing = 9 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	34	7.827	20.988	5484	6441	f(t) Total Load
	4987	100	44	8.987	24.110	6435	8075	f(t) Total Load
	5862	108.9	56	9.893	27.401	7338	9862	f(t) Total Load *
8000	6000	90	32	6.591	19.231	5526	6078	f(t) Total Load
	6000	100	44	8.358	23.689	6282	8042	f(t) Total Load
	6088	110	58	9.970	27.906	7500	10082	f(t) Total Load
	7087	118.2	78	10.655	31.557	8381	12198	f(t) Total Load [e]
10000	7500	90	32	6.051	18.843	5558	6078	f(t) Total Load
	7500	100	42	7.335	22.203	6518	7725	f(t) Total Load
	7500	120	56	8.931	26.600	7541	9809	f(t) Total Load
	7500	118.7	78	10.399	31.321	8483	12198	f(t) Total Load [e]
12000	9000	90	32	5.647	18.553	5585	6078	f(t) Total Load
	9000	100	42	6.834	21.864	6548	7725	f(t) Total Load
	9000	110	54	8.077	25.393	7574	9534	f(t) Total Load
	9000	119.5	78	9.646	30.744	8611	12199	f(t) Total Load [e]

Table D.19. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 4.25 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	14	4.448	13.649	3598	3822	Ultimate Moment Strength/f(t)_T L
	4500	100	18	5.372	15.663	4247	4790	f(t) Total Load
	4500	110	24	6.933	19.282	4943	6169	f(t) Total Load
	4500	120	28	7.764	20.491	5685	7010	f(t) Total Load
	5680	129.5	34	8.043	22.700	6437	8166	f(t) Total Load *
8000	6000	90	14	4.023	13.318	3621	3808	Ultimate Moment Strength/f(t)_T L
	6000	100	18	4.837	15.262	4273	4766	f(t) Total Load
	6000	110	22	5.574	16.944	4971	5684	f(t) Total Load
	6000	120	28	6.779	19.957	5716	6960	f(t) Total Load
	6000	130	34	7.846	22.547	6509	8115	f(t) Total Load
	6000	140	42	8.860	25.878	7348	9460	f(t) Total Load
	7682	147.8	50	9.612	29.048	8040	10600	f(t) Total Load *
10000	7500	90	14	3.729	13.089	3638	3808	Ultimate Moment Strength/f(t)_T L
	7500	100	18	4.466	14.983	4293	4766	f(t) Total Load
	7500	110	22	5.135	16.626	4993	5684	f(t) Total Load
	7500	120	28	6.231	19.583	5741	6964	f(t) Total Load
	7500	130	34	7.204	22.134	6535	8128	f(t) Total Load
	7500	140	42	8.483	25.646	7377	9504	f(t) Total Load
	8027	150	52	9.708	29.680	8267	10961	f(t) Total Load
	9182	160	70	11.280	36.180	9204	12693	f(t) Total Load
	9335	160.8	74	11.496	37.250	9283	12827	f(t) Total Load/f(c)_T D L
12000	9000	90	14	3.510	12.918	3653	3808	Ultimate Moment Strength/f(t)_T L
	9000	100	18	4.188	14.774	4309	4766	Ultimate Moment Strength/f(t)_T L
	9000	110	22	4.806	16.388	5012	5684	f(t) Total Load
	9000	120	26	5.330	17.673	5761	6554	f(t) Total Load
	9000	130	34	6.721	21.822	6558	8137	f(t) Total Load
	9000	140	42	7.908	25.303	7401	9533	f(t) Total Load
	9000	150	52	9.288	29.449	8293	11032	f(t) Total Load
	9182	160	70	11.280	36.173	9232	12881	f(t) Total Load
	9590	161.6	76	11.483	37.579	9384	13110	f(t) Total Load [e]

Table D.20. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	16	5.219	15.405	3910	4360	Ultimate Moment Strength/f(t)_T L
	4500	100	20	6.120	17.300	4608	5334	f(t) Total Load
	4500	110	26	7.594	20.653	5356	6714	f(t) Total Load
	5252	120	32	8.332	23.083	6153	7993	f(t) Total Load
	5930	129.6	38	8.966	25.241	6965	9141	f(t) Total Load *
8000	6000	90	16	4.699	15.009	3936	4344	Ultimate Moment Strength/f(t)_T L
	6000	100	20	5.495	16.842	4637	5309	f(t) Total Load
	6000	110	24	6.217	18.416	5388	6240	f(t) Total Load
	6000	120	32	7.915	22.806	6189	7936	f(t) Total Load
	6000	130	38	8.900	25.113	7039	9083	f(t) Total Load
	7307	140	48	9.866	29.208	7939	10726	f(t) Total Load
	7994	143.8	52	10.079	30.600	8294	11291	f(t) Total Load *
10000	7500	90	16	4.339	14.735	3956	4344	Ultimate Moment Strength
	7500	100	20	5.061	16.523	4660	5309	f(t) Total Load
	7500	110	24	5.717	18.064	5414	6240	f(t) Total Load
	7500	120	30	6.751	20.785	6216	7527	f(t) Total Load
	7500	130	38	8.165	24.659	7069	9095	f(t) Total Load
	7500	140	48	9.767	29.145	7972	10779	f(t) Total Load
	8836	150	62	10.990	34.463	8925	12619	f(t) Total Load
	9421	155.0	76	11.852	38.420	9417	13551	f(t) Total Load / f(c)_T D L
12000	9000	90	16	4.071	14.530	3973	4344	Ultimate Moment Strength
	9000	100	18	4.188	14.472	4679	4831	Ultimate Moment Strength/f(t)_T L
	9000	110	24	5.343	17.799	5435	6240	f(t) Total Load
	9000	120	30	6.299	20.483	6239	7527	f(t) Total Load
	9000	130	38	7.611	24.316	7094	9103	f(t) Total Load
	9000	140	46	8.730	27.515	7999	10502	f(t) Total Load
	9000	150	60	10.624	33.419	8954	12494	f(t) Total Load
	9423	155.4	76	11.833	38.335	9487	13764	f(t) Total Load [e]

Table D.21. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 5.75 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	18	5.973	17.116	4215	4897	f(t) Total Load
	4500	100	22	6.853	18.899	4962	5876	f(t) Total Load
	4500	110	28	8.242	21.991	5761	7253	f(t) Total Load
	5641	120	34	8.674	24.121	6612	8559	f(t) Total Load
	5979	125.0	38	9.223	25.849	7057	9360	f(t) Total Load *
8000	6000	90	16	4.699	14.761	4244	4382	Ultimate Moment Strength/f(t)_T L
	6000	100	22	6.142	18.389	4995	5850	f(t) Total Load
	6000	110	28	7.375	21.404	5797	7211	f(t) Total Load
	6000	120	34	8.469	23.984	6652	8500	f(t) Total Load
	7012	130	44	9.752	28.530	7559	10384	f(t) Total Load
	7967	138.6	52	10.377	31.297	8384	11657	f(t) Total Load *
10000	7500	90	16	4.339	14.487	4267	4382	Ultimate Moment Strength/f(t)_T L
	7500	100	20	5.061	16.221	5020	5369	Ultimate Moment Strength/f(t)_T L
	7500	110	26	6.247	19.363	5826	6772	f(t) Total Load
	7500	120	34	7.768	23.535	6683	8500	f(t) Total Load
	7500	130	42	9.061	27.002	7593	10041	f(t) Total Load
	8305	140	54	10.506	32.087	8555	12012	f(t) Total Load
	9393	149.4	76	12.110	39.044	9513	14143	f(t) Total Load [e]
12000	9000	90	16	4.071	14.282	4285	4382	Ultimate Moment Strength/f(t)_T L
	9000	100	20	4.736	15.983	5041	5369	Ultimate Moment Strength/f(t)_T L
	9000	110	26	5.832	19.076	5849	6772	f(t) Total Load
	9000	120	32	6.773	21.643	6709	8069	f(t) Total Load
	9000	130	42	8.441	26.633	7621	10048	f(t) Total Load
	9000	140	54	10.182	31.912	8586	12053	f(t) Total Load
	9395	149.8	76	12.093	38.966	9583	14317	f(t) Total Load [e]

Table D.22. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 7 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	20	6.713	18.623	4713	5458	f(t) _ Total Load
	4500	100	26	8.223	21.969	5539	6920	f(t) _ Total Load
	5216	110	32	8.972	24.354	6422	8289	f(t) _ Total Load
	5883	118.2	38	9.686	26.781	7187	9587	f(t) _ Total Load *
8000	6000	90	18	5.363	16.249	4746	4938	Ultimate Moment Strength/f(t)_T L
	6000	100	24	6.778	19.705	5577	6426	f(t) _ Total Load
	6000	110	32	8.495	24.041	6464	8245	f(t) _ Total Load
	6250	120	40	9.835	27.625	7404	9928	f(t) _ Total Load
	7925	130	52	10.853	32.526	8407	12141	f(t) _ Total Load
	7930	131.1	52	10.796	32.277	8517	12141	f(t) _ Total Load *
10000	7500	90	18	4.939	15.933	4773	4938	Ultimate Moment Strength/f(t)_T L
	7500	100	24	6.223	19.316	5607	6426	f(t) _ Total Load
	7500	110	30	7.282	22.003	6497	7799	f(t) _ Total Load
	7500	120	40	9.159	27.216	7444	9928	f(t) _ Total Load
	7593	130	50	10.657	31.407	8447	11818	f(t) _ Total Load
	9209	140	70	12.218	38.602	9508	14555	f(t) _ Total Load
	9630	141.5	76	12.273	39.810	9667	14956	f(t) _ Total Load [e]
12000	9000	90	18	4.622	15.697	4795	4938	Ultimate Moment Strength/f(t)_T L
	9000	100	24	5.808	19.023	5632	6426	f(t) _ Total Load
	9000	110	30	6.788	21.675	6525	7799	f(t) _ Total Load
	9000	120	38	8.117	25.449	7474	9526	f(t) _ Total Load
	9000	130	50	9.971	31.030	8480	11826	f(t) _ Total Load
	9209	140	70	12.218	38.597	9544	14637	f(t) _ Total Load
	9613	141.8	76	12.321	39.768	9740	15079	f(t) _ Total Load [e]

Table D.23. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 8.5 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	22	7.439	20.014	5294	6025	f(t) _ Total Load
	4794	100	30	9.526	24.538	6214	7961	f(t) _ Total Load
	5856	111.3	38	10.095	27.660	7322	9779	f(t) _ Total Load *
8000	6000	90	22	6.655	19.455	5333	6007	f(t) _ Total Load
	6000	100	28	7.919	22.412	6258	7460	f(t) _ Total Load
	6000	110	38	10.066	27.885	7244	9727	f(t) _ Total Load
	7215	120	48	11.005	31.816	8292	11772	f(t) _ Total Load
	7939	123.4	52	11.175	33.210	8658	12544	f(t) _ Total Load *
10000	7500	90	22	6.109	19.065	5364	6007	f(t) _ Total Load
	7500	100	28	7.260	21.968	6293	7460	f(t) _ Total Load
	7500	110	36	8.779	25.968	7283	9299	f(t) _ Total Load
	7500	120	48	10.839	31.719	8335	11772	f(t) _ Total Load
	9016	130	64	12.139	37.660	9449	14509	f(t) _ Total Load
	9916	133.0	74	12.422	40.197	9794	15596	f(t) _ Total Load *[HD]
12000	9000	90	20	5.165	17.001	5390	5499	Ultimate Moment Strength/f(t)_T L
	9000	100	28	6.765	21.633	6322	7460	f(t) _ Total Load
	9000	110	36	8.173	25.590	7315	9299	f(t) _ Total Load
	9000	120	46	9.734	30.085	8370	11388	f(t) _ Total Load
	9000	130	64	12.115	37.571	9488	14510	f(t) _ Total Load
	10028	133.7	76	12.434	40.487	9911	15805	f(t) _ Total Load [e]

Table D.24. Type IV Beam Designs - AASHTO LRFD Specifications (Strand Diameter = 0.6 in., Girder Spacing = 9 ft.).

f'_c (psi)	f'_{ci} (psi)	Length (ft.)	No. Strands	Initial Loss (%)	Final Loss (%)	M_u (kip-ft.)	ϕM_n (kip-ft.)	Controlling Limit State
6000	4500	90	24	8.152	21.700	5484	6552	f(t) _ Total Load
	4595	100	30	9.412	24.444	6435	7992	f(t) _ Total Load
	5849	109.2	38	10.211	27.913	7363	9829	f(t) _ Total Load *
8000	6000	90	22	6.655	19.291	5526	6025	Ultimate Moment Strength/f(t)_T L
	6000	100	30	8.476	23.825	6482	7961	f(t) _ Total Load
	6211	110	40	10.402	28.965	7500	10208	f(t) _ Total Load
	7879	120	52	11.369	33.745	8583	12621	f(t) _ Total Load
	7884	121.0	52	11.317	33.502	8699	12621	f(t) _ Total Load *
10000	7500	90	22	6.109	18.901	5558	6025	Ultimate Moment Strength/f(t)_T L
	7500	100	30	7.767	23.356	6518	7961	f(t) _ Total Load
	7500	110	38	9.221	27.138	7541	9780	f(t) _ Total Load
	7548	120	50	11.189	32.651	8627	12240	f(t) _ Total Load
	9895	130	72	12.433	40.047	9778	15630	f(t) _ Total Load
	9888	130.1	72	12.432	40.021	9793	15630	f(t) _ Total Load *[HD]
12000	9000	90	22	5.701	18.609	5585	6025	Ultimate Moment Strength/f(t)_T L
	9000	100	28	6.765	21.436	6548	7489	f(t) _ Total Load
	9000	110	38	8.583	26.747	7574	9780	f(t) _ Total Load
	9000	120	50	10.441	32.241	8664	12240	f(t) _ Total Load
	9895	130	72	12.433	40.044	9818	15667	f(t) _ Total Load
	10148	131.2	76	12.467	40.695	9963	16012	f(t) _ Total Load [e]

Table D.25. Controlling Limit States and Maximum Span Lengths for $f_t = 6\sqrt{f'_c}$ and $f_t = 7.5\sqrt{f'_c}$ (Type IV Beams - AASHTO LRFD Specifications - Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	$f_t = 6\sqrt{f'_c}$				$f_t = 7.5\sqrt{f'_c}$				% Diff. Max. Span	Diff. No. Strands
		f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State	f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State		
6000	4.25	5680	129.54	34	f (t) _Total Load *	5691	132.01	34	f (t) _TL *	1.9	0
	5.00	5930	129.60	38	f (t) _Total Load *	5940	131.94	38	f (t) _T L *	1.8	0
	5.75	5979	125.00	38	f (t) _Total Load *	5920	127.26	38	f (t) _T L *	1.8	0
	7.00	5883	118.19	38	f (t) _Total Load *	5892	120.37	38	f (t) _T L *	1.8	0
	8.50	5856	111.3	38	f (t) _Total Load *	5864	113.32	38	f (t) _T L *	3.6	0
	9.00	5849	109.18	38	f (t) _Total Load *	5856	111.2	38	f (t) _T L *	1.9	0
8000	4.25	7682	147.84	50	f (t) _Total Load *	7696	150.53	50	f (t) _T L *	1.8	0
	5.00	7994	143.80	52	f (t) _Total Load *	8000	146.37	52	f (t) _T L *	1.8	0
	5.75	7967	138.63	52	f (t) _Total Load *	7980	141.14	52	f (t) _T L *	1.8	0
	7.00	7930	131.06	52	f (t) _Total Load *	7942	133.46	52	f (t) _T L *	1.8	0
	8.50	7939	123.36	52	f (t) _Total Load *	7904	125.62	52	f (t) _T L *	1.8	0
	9.00	7884	121.04	52	f (t) _Total Load *	7894	123.3	52	f (t) _T L *	1.8	0
10000	4.25	9335	160.82	74	f (t)_TL / f (c) _TDL	9198	162.79	70	f (t)_TL / f (c) _TDL	1.2	0
	5.00	9421	154.97	76	f (t)_TL / f (c) _TDL	9626	157.02	72	f (t)_TL / f (c) _TDL	1.3	0
	5.75	9393	149.44	76	f (t) _TL [e]	9288	151.77	74	f (t)_TL / f (c) _TDL	1.6	0
	7.00	9630	141.45	76	f (t) _TL [e]	9500	143.92	76	f(t) _ TL [e] / f(c)_TDL	1.7	0
	8.50	9916	132.99	74	f (t) _TL [e]	9926	135.69	76	f(t) _TL [e]	2.0	0
	9.00	9888	130.13	72	f (t) _TL *	9922	132.9	74	f (t)_TL / f (c) _TDL	2.1	0
12000	4.25	9590	161.59	76	f (t) _TL [e]	9471	164.61	76	f (t) _TL [e]	1.9	0
	5.00	9423	155.36	76	f (t) _TL [e]	9000	158.33	76	f (t) _TL [e]	1.9	0
	5.75	9395	149.80	76	f (t) _TL [e]	9419	152.69	76	f (t) _TL [e]	1.9	0
	7.00	9613	141.78	76	f (t) _TL [e]	9470	144.47	76	f (t) _TL [e]	1.9	0
	8.50	10028	133.65	76	f (t) _TL [e]	9899	136.22	76	f (t) _TL [e]	1.9	0
	9.00	10148	131.22	76	f (t) _TL [e]	10023	133.7	76	f (t) _TL [e]	1.9	0

Table D.26 Controlling Limit States and Maximum Span Lengths for $f_t = 6\sqrt{f'_c}$ and $f_t = 8\sqrt{f'_c}$ (Type IV Beams - AASHTO LRFD Specifications - Strand Diameter = 0.6 in.).

f'_c (psi)	Girder Spacing (ft.)	$f_t = 6\sqrt{f'_c}$ f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State	$f_t = 8\sqrt{f'_c}$ f'_{ci} (psi)	Max. Span (ft.)	No. Strands	Controlling Limit State	% Diff. Max. Span	Diff. No. Strands
6000	4.25	5680	129.54	34	f (t) _Total Load *	5693	132.8	34	f (t) _TL *	2.5	0
	5.00	5930	129.60	38	f (t) _Total Load *	5943	132.6	38	f (c) _T L *	2.3	0
	5.75	5979	125.00	38	f (t) _Total Load *	5923	128.0	38	f (t) _T L *	2.4	0
	7.00	5883	118.19	38	f (t) _Total Load *	5894	121.1	38	f (t) _T L *	2.5	0
	8.50	5986	109.36	36	f (t) _Total Load *	5866	114.0	38	f (t) _T L *	4.2	0
	9.00	5849	109.18	38	f (t) _Total Load *	5859	111.9	38	f (t) _T L *	2.5	0
8000	4.25	7682	147.84	50	f (t) _Total Load *	7700	151.4	50	f (t) _T L *	2.4	0
	5.00	7994	143.80	52	f (t) _Total Load *	8000	147.3	52	f (t) _T L *	2.4	0
	5.75	7967	138.63	52	f (t) _Total Load *	7984	142.0	52	f (t) _T L *	2.4	0
	7.00	7930	131.06	52	f (t) _Total Load *	7945	134.3	52	f (t) _T L *	2.4	0
	8.50	7939	123.36	52	f (t) _Total Load *	7907	126.4	52	f (t) _T L *	2.4	0
	9.00	7884	121.04	52	f (t) _Total Load *	7897	124.0	52	f (t) _T L *	2.5	0
10000	4.25	9335	160.82	74	f (t)_TL / f (c) _TDL	9199	163.1	70	f (c) _TDL	1.4	0
	5.00	9421	154.97	76	f (t)_TL / f (c) _TDL	9230	157.7	72	f (c) _TDL	1.8	0
	5.75	9393	149.44	76	f (t) _TL [e]	9291	152.6	74	f (c) _TDL	2.1	0
	7.00	9630	141.45	76	f (t) _TL [e]	9455	144.7	76	f(c)_TDL	2.3	0
	8.50	9916	132.99	74	f (t) _TL [e]	9887	136.5	76	f(t) _TL [e]	2.6	0
	9.00	9888	130.13	72	f (t) _TL *	10000	134.0	76	f (t) _T L *	3.0	0
12000	4.25	9590	161.59	76	f (t) _TL [e]	9476	165.6	76	f (t) _TL [e]	2.5	0
	5.00	9423	155.36	76	f (t) _TL [e]	9444	159.3	76	f (t) _TL [e]	2.5	0
	5.75	9395	149.80	76	f (t) _TL [e]	9415	153.6	76	f (t) _TL [e]	2.6	0
	7.00	9613	141.78	76	f (t) _TL [e]	9423	144.3	76	f (t) _TL [e]	1.8	0
	8.50	10028	133.65	76	f (t) _TL [e]	9857	137.1	76	f (t) _TL [e]	2.6	0
	9.00	10148	131.22	76	f (t) _TL [e]	9983	134.6	76	f (t) _TL [e]	2.6	0