Texas Transportation Institute

Impact of LRFD Specifications on the Design of Texas Bridges

A Seminar for TxDOT Engineers
Sponsored by the Texas Department of Transportation
TxDOT Research Project 0-4751

August 29, 2005
College Station, Texas

Texas Transportation Institute
The Texas A&M University System
College Station, Texas

Texas Department of Transportation

NOTICE

The material presented herein is intended for instructional purposes only. Much of the material is in final draft form as the date of the seminar precedes the completion date of the research study and submittal of the final report for approval. This material is not meant as a substitute for the actual design codes and specifications during the design of prestressed highway bridge girders.

Table of Contents

Impact of LRFD Specifications on the Design of Texas Bridges
Monday, August 29, 2005
College Station Hilton & Conference Center
801 University Drive East, College Station, Texas

Seminar Description

This one-day seminar is part of TxDOT research project 0-4751 "Impact of LRFD Specifications on the Design of Texas Bridges." The seminar will highlight significant differences in the *AASHTO LRFD Bridge Design Specifications* as compared to the *AASHTO Standard Specifications for Highway Bridges* with a focus on provisions that affect the design of typical prestressed concrete bridges in Texas and associated substructure elements. Detailed design examples will focus on the application of the LRFD specifications to prestressed concrete superstructure and substructure design.

Seminar Agenda

Time	Description	Speaker
8:30 - 9:15	**Registration**	
9:15 - 9:30	**Welcome and Introductory Remarks**	M. Hueste D. Christiansen D. Rosowsky
9:30 - 9:45	**TxDOT LRFD Implementation**	R. Ruperto G. Freeby
9:45 – 10:30	**Introduction to the AASHTO LRFD Bridge Design Specifications** • Introduction to Reliability Theory and Calibration of AASHTO LRFD Specifications • Overview of New Concepts Used in the LRFD Specifications	D. Mertz
10:30 - 10:45	**Break**	
10:45 - 12:00	**Prestressed Concrete Superstructure Design** • Critical Differences from Standard Specifications • Impact of LRFD Specifications on Typical Texas Bridges – Parametric Study	M. Hueste P. Keating M. Adil M. Adnan
12:00 - 1:00	**Lunch**	
1:00 - 1:45	**Prestressed Concrete Superstructure Design, cont.** • Application of the LRFD Specifications: Prestressed Concrete Bridge Girder Design Example	M. Hueste M. Adil
1:45 - 2:30	**Substructure Analysis and Design** • Critical Differences from Standard Specifications • Impact of LRFD Specifications on Typical Texas Bridges – Parametric Study	M. Diaz E. Ingamells
2:30 - 2:45	**Break**	
2:45 – 3:15	**Substructure Analysis and Design, cont.** • Application of the LRFD Specifications: Substructure Design Example	M. Diaz E. Ingamells
3:15 - 3:30	**Transitioning to LRFD - Design Issues and Recommendations**	M. Hueste D. Mertz
3:30 - 3:45	**Concluding Remarks, Evaluation Forms, and CEU Certificates**	M. Hueste

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Manuel A. Diaz, Ph.D., P.E.
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Mary Beth D. Hueste, Ph.D., P.E.
Dr. Hueste is an Assistant Professor in the Civil Engineering Department and an Assistant Research Engineer with the Texas Transportation Institute (TTI), both at Texas A&M University. She also serves as the Structures Program Manager for the Constructed Facilities Division of TTI. Dr. Hueste's research is focused on design and evaluation of prestressed concrete bridge structures and earthquake resistant design of concrete structures. She teaches undergraduate and graduate courses in structural engineering, including reinforced and prestressed concrete design. Dr. Hueste holds a B.S. degree from North Dakota State University, a M.S. degree from the University of Kansas, and a Ph.D. degree from the University of Michigan; all in Civil Engineering. She is a registered professional engineer in Kansas and Texas.
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Peter B. Keating, Ph.D.
Dr. Keating is an Associate Professor in the Civil Engineering Department and an Associate Research Engineer with the Texas Transportation Institute, both at Texas A&M University. He was awarded his Bachelor of Science, Bachelor of Arts, Master of Science, and Doctor of Philosophy degrees all from Lehigh University in Bethlehem, Pennsylvania. Dr. Keating teaches both undergraduate and graduate courses in structural analysis and design. Dr. Keating's general area of interest is in the fatigue behavior of welded structures with specific interest in high cycle or extreme-life fatigue and the deleterious effects of overloads. Dr. Keating co-authored revisions to the fatigue provisions contained in the specifications of both the American Institute of Steel Construction and the American Association of State Highway and Transportation Officials.
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Dennis R. Mertz, Ph.D., P.E.
Professor Mertz teaches bridge engineering at the University of Delaware, and is the Director of the University's Center for Innovative Bridge Engineering (CIBrE). Previous to his appointment to the University, he was an Associate of the bridge design firm of Modjeski & Masters, Inc. Dennis was the Co-Principal Investigator of the NCHRP research project which wrote the original edition of the AASHTO *LRFD Bridge Design Specifications*. He continues to be active in its further development and implementation. All of Professor Mertz's engineering degrees are from Lehigh University in Bethlehem, Pennsylvania. He is also a Professional Engineer in the Commonwealth of Pennsylvania.
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Graduate Students

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Mr. Adil was born in Hyderabad, India. He received his Bachelor's in Civil Engineering from Osmania University, India. After working as a Structural Engineer for one year in India, he moved to Saudi Arabia to pursue graduate studies. He worked towards his Master's in Structural Engineering for one year at King Fahd University of Petroleum & Minerals (KFUPM) before moving to Texas A&M University. At Texas A&M University he is enrolled as a Master's student in Civil Engineering (structures emphasis). He has been conducting research with Dr. Mary Beth Hueste and Dr. Peter Keating to evaluate the impact of the AASHTO LRFD Specifications on prestressed concrete bridges in Texas.
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Mohsin Adnan, B.S.
Mr. Adnan was born in Peshawar, Pakistan. He studied in NWFP University of Engineering and Technology, Pakistan, where he received his Bachelor's degree in Civil Engineering in 2001. He worked in two different consulting firms as a Design Engineer for two years and as a Lecturer in NWFP University for four months before moving to Texas A&M University. At Texas A&M University he is enrolled as a Master's student in Civil Engineering (structures emphasis). He has been working as a research assistant with Dr. Mary Beth Hueste and Dr. Peter Keating and his research focused on evaluating the impact of the AASHTO LRFD Specifications on prestressed concrete bridges in Texas.
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Eric R. Ingamells, B.A., B.S., P.E.
Mr. Ingamells is a bridge design engineer for the state of Texas. He is currently pursuing a Master's degree in Civil Engineering at the University of Texas at San Antonio (UTSA). Prior to his pursuit of higher education he was engaged for nearly a decade in bridge design, analysis, widening rehab, load-rating, overloads, and inspection, etc.; while working in the Texas Department of Transportation's Bridge Division in Austin, Texas.
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Impact of LRFD Specifications on the Design of Texas Bridges

TxDOT Research Project 0-4751 - Fact Sheet

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Manuel Diaz (UTSA)
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Graduate Students:
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TxDOT Personnel:
Rachel Ruperto (Project Director)
David Hohmann (Program Coordinator)

Project Duration: September 2003 – August 2005

Project Summary

The *AASHTO Standard Specifications for Highway Bridges* will no longer be updated and TxDOT intends to transition to the use of the *AASHTO LRFD Bridge Design Specifications*, as required for all bridges receiving federal funding by 2007. The LRFD Specifications include significant changes for the design of bridges for both demand and capacity. The purpose of this project is to evaluate the impact of these provisions on the design of typical Texas bridges.

The project objectives are met through a series of seven tasks: (1) Review literature and current state of practice, (2) Define prototype Texas bridges, (3) Develop detailed design examples, (4) Conduct parametric study, (5) Identify and address needs for revised design criteria, (6) Complete final reports and recommendations, and (7) Plan and conduct seminar.

The TTI research team at Texas A&M University has focused their efforts on bridge girder design. Two sets of parallel detailed design examples for bridge girders have been developed as instructional materials for use by TxDOT, using parameters representative of typical bridges in Texas. Type IV and U54 girders are used in each set of parallel examples, where the first design in a set follows the Standard Specifications and the second design follows the LRFD Specifications. In addition to the requirements of the specifications, typical TxDOT design practices are implemented in the examples when possible.

A parametric study was also conducted to further evaluate the impact of the LRFD design criteria on typical Texas bridges as compared to the Standard Specifications. Three prestressed concrete girder types were considered: Type C, Type IV and U54. Additional parameters that were varied include span lengths, girder spacings, strand diameter, and skew angle. The concrete strength at release and service were optimized based on TxDOT practice. The parametric study identifies limitations of the new LRFD criteria and areas within the design most impacted by the transition to the LRFD Specifications.

Additional research conducted at the University of Texas at San Antonio is focused on typical Texas bridge substructures. This research will produce a detailed design example demonstrating the application of the LRFD Specifications to substructure components. A parametric study will demonstrate the impact of the LRFD Specifications on the design of bridge substructures for typical Texas bridges.

The results of this study will be disseminated to TxDOT engineers through a seminar in August 2005. In addition, two project reports containing the detailed examples, details of the parametric study, and research findings will be available in late 2005. Finally, conference and journal papers will be developed to disseminate the research findings to the professional community.

Project Contact: For additional information, please contact Dr. Mary Beth Hueste
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Introduction to the LRFD Bridge Design Specifications

Dennis Mertz
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Design Methods

- Service Load Design (SLD)
  (Allowable Stress Design, ASD; or Working Stress Design, WSD))
- Strength Design Method
  (Load Factor Design, LFD)
- Load and Resistance Factor Design (LRFD)

Design Methodology Evolution

Service Load (SLD)	$f_t \le 0.55F_y$
	$(f\ )_{DL} + (f_t)_{LL} \le 0.55F_y$
	$(f_t)_{DL} + (f_t)_{LL} \le \frac{1}{1.82}F_y$
	$1.82(f_t)_{DL} + 1.82(f_t)_{LL} \le F_y$
Load Factor (LFD)	$1.3(f_t)_{DL} + 2.17(f_t)_{LL} \le F_y$
Load and Resistance Factor (LRFD)	$1.25(f_t)_{DL} + 1.75(f_t)_{LL} \le F_y$

Evolution of Methodologies

- SLD
  - Linear elastic stress-strain distribution
  - $f_c \le 0.40\ f_c$'
  - $f_y \le 24$ ksi (Grade 60)
- LFD & LRFD
  - Non-linear stress-strain (equivalent rectangular stress block)
  - Tension steel yields before concrete crushes → ductile behavior

AASHTO Bridge Design Specifications

- Standard Specifications for Highway Bridges, 17th Edition, 2002
- AASHTO LRFD Bridge Design Specifications,
  - Investigation begun in 1986
  - Development begun in 1988
  - 1st Edition, 1994
  - 2nd Edition, 1998
  - 3rd Edition, 2004
  - Available in US and SI Units

AASHTO Ballots on LRFD

May 1993

"To adopt the final draft of the NCHRP 12-33 document as the 1993 LRFD Specifications for Highway Bridge Design and in 1995 consider phasing out the current Standard Specifications."

May 1999

"After the 1999 meeting, discontinue maintenance of the Standard Specifications (except to correct errors), and maintain the LRFD Specifications."

AASHTO Recommendation - LRFD Implementation Plan (2000)

- All new bridges on which States initiate preliminary engineering after October 1, 2007, shall be designed by the LRFD Specifications
- States unable to meet these dates will provide justification and a schedule for completing the transition to LRFD.
- For modifications to existing structures, States would have the option of using LRFD Specifications or the specifications which were used for the original design.

Objective of LRFD

Develop a comprehensive and consistent Load and Resistance Factor Design (LRFD) specification that is calibrated to obtain uniform reliability (a measure of safety) at the strength limit state for all materials.

CALIBRATION

Selection of a set of γ's and ϕ's to approximate a target level of reliability in an LRFD-format specification.

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What's <u>not</u> LRFD?

- New limit states,
- New, more complex live-load distribution factors,
- New unified-concrete shear design using modified compression-field theory,
- Strut-and-tie model for concrete, and
- Many other state-of-the-art additions.

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Limit States

- Service limit states,
- Fatigue-and-fracture limit states,
- Strength limit states, and
- Extreme-event limit states.

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Only the strength limit states of the *LRFD Specifications* are calibrated based upon the theory of structural reliability, wherein statistical load and resistance data are required.

The other limit states are based upon the design criteria of the *Standard Specifications*.

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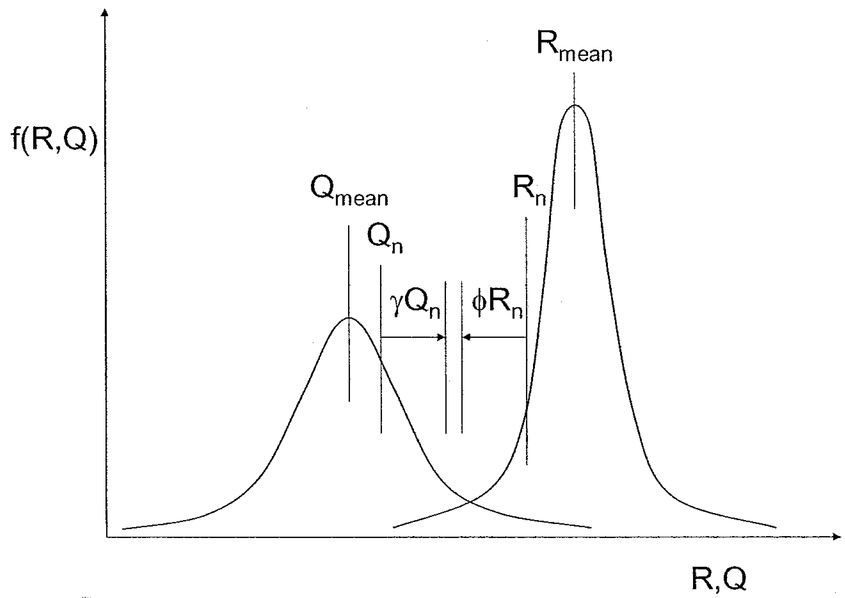
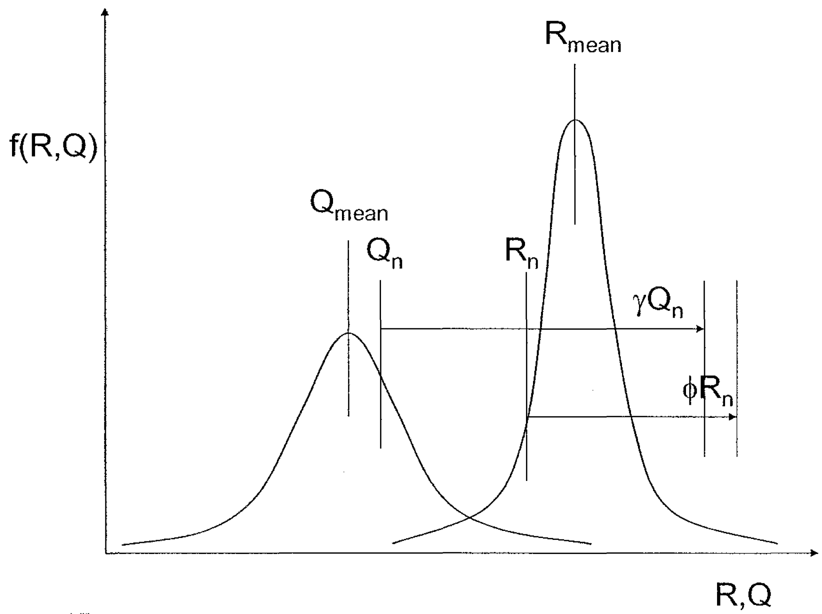


THE TARGET RELIABILITY INDEX β IS A UNIQUE QUANTITY.

Many different sets of γ's and ϕ's can be selected to achieve the unique reliability index β.



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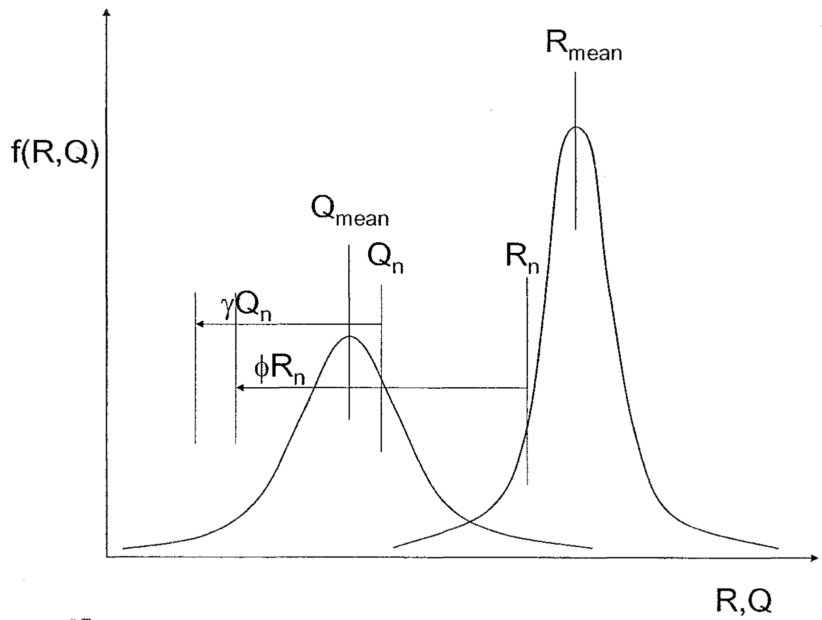

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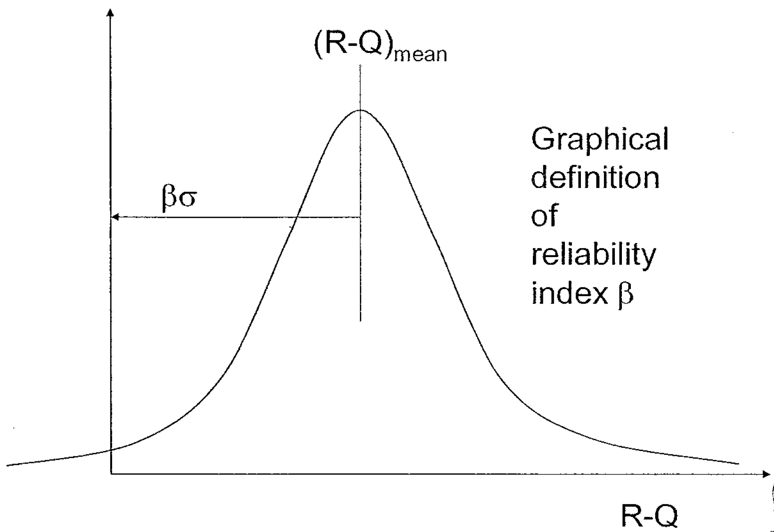

Graphical definition of reliability index β

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What is an acceptable value for β?

Can we examine human behavior to choose a target β for bridge design?

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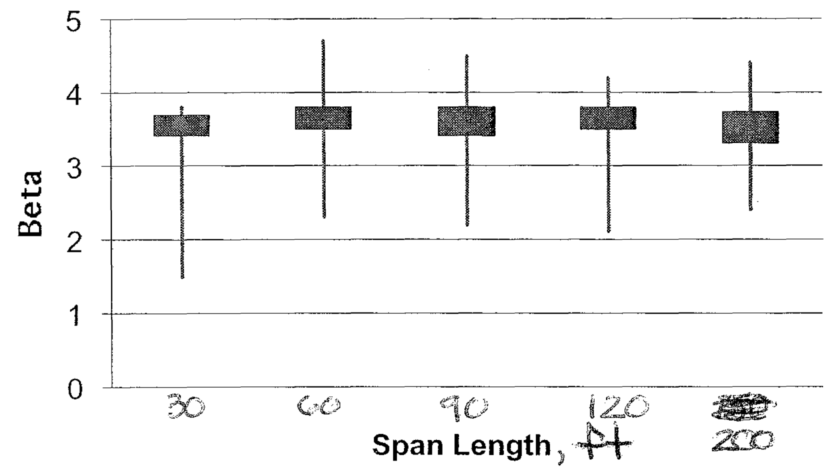
Reliability Indices



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If load and resistance are normal random variables,

$$\sigma_{(R-Q)} = \sqrt{\sigma_R^{\;2} + \sigma_Q^{\;2}}$$

and

$$\beta = \frac{R_{mean} - Q_{mean}}{\sqrt{\sigma_R^{\;2} + \sigma_Q^{\;2}}}$$

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Impact of LRFD on TX Bridges

LRFD requires that:

$$\phi R \geq \sum_i \gamma_i Q_i$$

And the nominal design resistance is defined as:

$$R_n = \frac{R_{mean}}{\lambda}$$

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From the definitions of β and λ

$$R_{mean} = Q_{mean} + \beta\sqrt{\sigma_R^{\ 2} + \sigma_Q^{\ 2}} = \lambda R_n$$

but

$$\phi R_n \geq \sum_i \gamma_i Q_i$$

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Finally, solving for
ϕ yields

$$\phi = \frac{\lambda_R \sum_i \gamma_i Q_i}{Q_{mean} + \beta\sqrt{\sigma_R^{\ 2} + \sigma_Q^{\ 2}}}$$

With three "unknowns," ϕ, the γ_i's and β

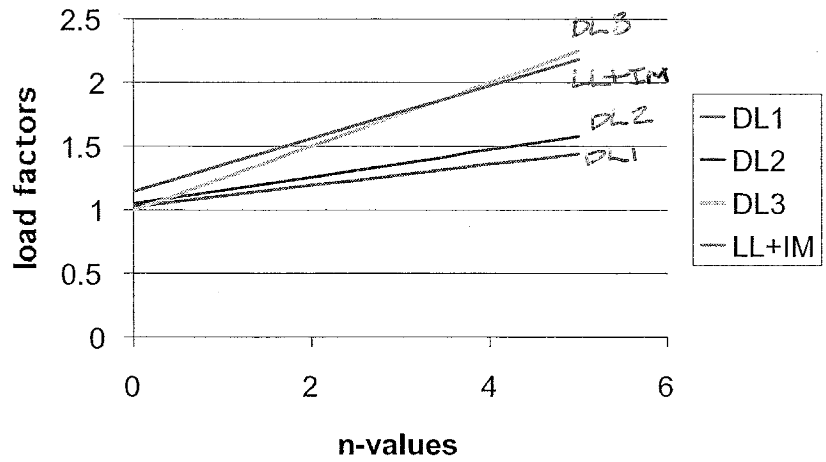
Load factors can be chosen such that all of the factored loads have an equal probability of being exceeded.

In equation form,

$$\gamma_i = \lambda_i(1 + nV_i)$$

where n is a constant for all load components.

Load Factors



With the target β and the γ's chosen, the ϕ's to achieve the approximate desired level of reliability can be determined.

The process is repeated until a set of γ's and ϕ's agreeable to the codewriters is obtained.

After much investigation, it was determined that:

- the total load, Q, can be accurately assumed to be a normal random variable, and
- the resistance, R, can be accurately assumed to be a lognormal random variable.

Nowak's equation D-25 (adapted)

$$\beta = \frac{R_n \lambda_n (1 - n V_R)[1 - \ln(1 - n V_R)] - Q_{mean}}{\sqrt{[R_n V_n \lambda_n (1 - n V_R)]^2 + \sigma_Q^2}}$$

but

$$R^* = \phi R_n = Q^* = \sum \gamma Q$$

and

$$R^* = R_{mean}(1 - n V_R) = \lambda_R R_n (1 - n V_R) = \phi R_n$$

Thus, the calibration of the *LRFD Specifications* became a huge spreadsheet/bookkeeping iterative problem (see Nowak's Appendix F).

The calibration represented in the current edition of the *LRFD Specifications* was made in the late 1980's and early 1990's.

Today, calibration is done differently. Due to modern computer resources, calibration is done by simulation, Monte Carlo Simulation.

MONTE CARLO SIMULATION

•"Bins" of data are developed holding values of distributed loads and resistances.

•Values are extracted randomly, and the LRFD comparison is made, in other words, is factored resistance greater than or equal to factored load?

•Many, many such comparisons are made until the sampling allows the probability of failure, and thus β, to be determined.

A further complication is combinations of load.

In general, extreme load effects, such as a once-in-75 year live load and a once-in-75 year wind, have a low probability of occurring simultaneously.

This is reflected in the LRFD load combinations table.

Adaptation of LRFD Table 3.4.1-1

Combinations	DC	LL	wind
Strength I	1.25	1.75	-
Strength II	1.25	1.35	-
Strength III	1.25	-	1.40
Strength IV	1.5	-	-
Strength V	1.25	1.35	0.4

Permit Loads used for Design

Long Span 70% LL

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Impact of LRFD on TX Bridges

THE LRFD LIMIT STATES ARE CALIBRATED BASED UPON PAST PRACTICE.

The strength limit states are calibrated to achieve levels of reliability comparable to the *Standard Specifications*.

The service, and fatigue-and-fracture limit states are calibrated to achieve member proportions comparable to the *Standard Specifications*.

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Calibration consists of up to three steps:

- Reliability-based calibration,
- Calibration or comparison to past practice, and
- Liberal doses of engineering judgment.

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THE SERVICE LIMIT STATES GENERALLY GOVERN THE PROPORTIONS OF SUPERSTRUCTURE MEMBERS.

Positive-moment regions of steel girders are governed by the service II load combination.

Prestressed concrete members are governed by the service I or III load combinations.

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Impact of LRFD on TX Bridges

MANY QUESTIONS REMAIN TO BE ANSWERED.

- What is the appropriate β for bridge design and evaluation?
- Should all bridge components have the same β?
- Should all limit states have the same β?
- Is an "analysis factor" needed?

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Impact of LRFD on TX Bridges

New Live-Load Model Evolution

HS 20-44

↓

HTL-57

(similar to OHBDC truck)

↓

HL-93

(uses HS20 component loads)

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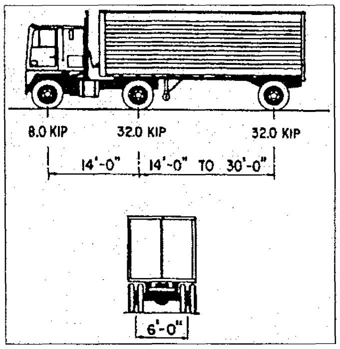
Impact of LRFD on TX Bridges

Design Vehicular Live Loads

Design Truck



Design Tandem

Two 25.0 KIP axles spaced 4.0 FT apart

Design Lane Load

Uniformly distributed load of 0.64 KLF

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Impact of LRFD on TX Bridges

Application of Design Vehicular LL

LRFD 3.6.1.2.1 and 3.6.1.3.1

Designation: HL-93

Service and Strength Limit States:

Design Truck OR Design Tandem
AND
Design Lane Load

The design lane load is not interrupted for the design truck or design tandem. Interruption is needed only where pattern loadings are used to produce maximum effects.

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Impact of LRFD on TX Bridges

Application of Design Vehicular Live Load

LRFD 3.6.1.3.1

Service and Strength Limit States:

Continuous Structures

For negative moment and reactions at interior piers, consider also the combination of

- 90% of the effect of two design trucks with a minimum of 50 FT between the rear axle of the lead truck and the front axle of the second truck. The spacing between 32 KIP axles on each truck shall be 14 FT.
- 90% of the effect of design lane load

Application of Design Vehicular Live Load

LRFD 3.6.1.4

Fatigue Limit State:

A Single Design Truck

The design truck shall have a constant spacing of 30 FT between 32 KIP axles.

Dynamic Load Allowance, IM

LRFD 3.6.2.1

Component	IM
Deck Joints - All Limit States	75%
All Other Components • Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

Applied only to the effects of the design truck or tandem – not to the design lane load

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Impact of LRFD on TX Bridges

Justification for New LL

New "notional" live load model simulates the shear and moment effects of a group of "exclusion" vehicles currently allowed to routinely travel on highways in various states.

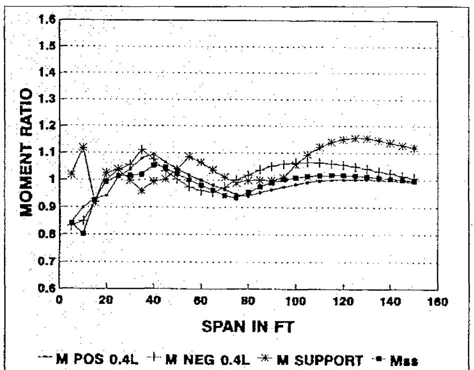

Figure C3.6.1.2.1-3 - Moment Ratios - Exclusion Vehicles to Notional Model

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Impact of LRFD on TX Bridges

Comparison of LL Effects LRFD Notional v/s HS20

The notional model produces live load moments and shears significantly greater than those caused by the HS20 loading especially for longer spans.

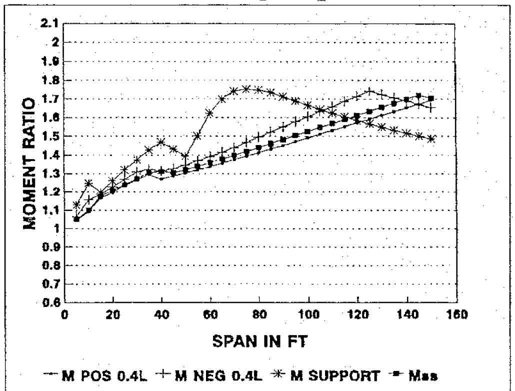

Figure C3.6.1.2.1-5 - Moment Ratios - Notional Model to HS20 (truck or lane) or two 24.0-KIP Axles at 4.0 FT

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Effect of New Design Vehicular LL

The total design load is also a function of the load factor, load modifier, load distribution and dynamic load allowance (impact).

This system of loads and factors was calibrated for the Strength Limit State to obtain uniform reliability for all materials

However, the Service Limit State usually governs the design of prestressed concrete members.

A special load combination for the Service Limit State was added to address this situation.

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Methods of Analysis

Refined Methods

- Classical Force and Displacement Methods
- Finite Element Method
- Finite Difference Method
- Grillage Analogy Method
- Others

Approximate Methods

- Distribution Factors

Approximate Methods of Analysis

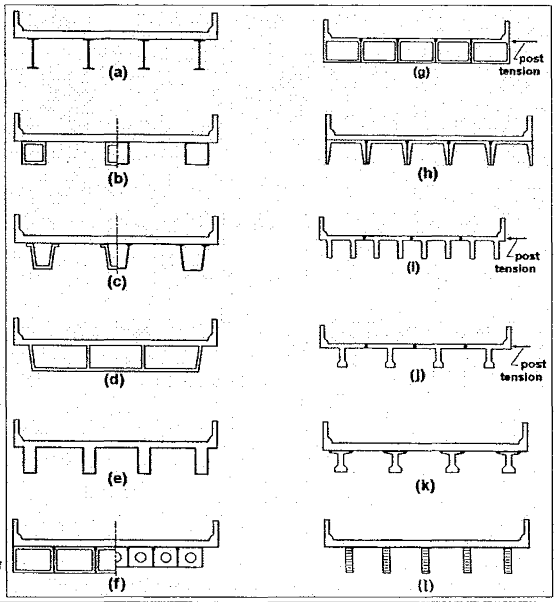
LRFD 4.6.2.2.1

General Limitations:

- Constant deck width
- Number of beams ≥ 4 (special provisions for 3 girders)
- Beams are parallel with equal stiffness
- Roadway part of overhang ≤ 3 ft
- Curvature in Plan is not less than limit in Article 4.6.1.2
- Cross-section appears in Table 4.6.2.2.1-1

Approximate Methods of Analysis

LRFD Table 4.6.2.2.1-1



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Types of Distribution and Correction Factors

Moment

- Section Types
- Interior Beams
- Exterior Beams
- Skewed Supports

Shear

- Section Types
- Interior Beams
- Exterior Beams
- Obtuse Corners

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Impact of LRFD on TX Bridges

For Moments – Interior Beams

Table 4.6.2.2.2b-1 Distribution of Live Loads Per Lane for Moment in Interior Beams.

Type of Beams	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Longitudinal vs. Transverse Stiffness

$$K_g = n\left(I + Ae_g^{\,2}\right)$$

n = modular ratio with deck concrete as base

I = moment of inertia of beam alone

A = are of beam

e_g = distance between deck mid-plane & beam centroid

Live-Load Distribution Factors

For Shear – Interior Beams

Table 4.6.2.2.3a-1 Distribution of Live Load per Lane for Shear in Interior Beams.

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T-and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$

CONCLUSIONS

The reliability-based LRFD design methodology is not perfect, but it represents an improvement over the ASD and LFD methodologies.

LRFD utilizes structural reliability to help us select improved load and resistance factors, and it provides a framework for future improvement.

CONCLUSIONS (continued)

Most of the features which designers dislike about the LRFD Specifications have little, if anything, to do with the LRFD design methodology.

Overview of Prestressed Concrete Superstructure Design

Mary Beth Hueste
Peter Keating
Mohammed Adil
Mohsin Adnan

Texas Transportation Institute

TxDOT LRFD Seminar
August 29, 2005 – College Station, Texas

Overview

- Outline of LRFD Concrete Section and Changes
  - Limit States
  - HL-93 Notional Load
  - Load Distribution
  - Debonding Limits
  - Prestress Losses
  - Shear Design by MCFT
  - Interface Shear Design Provisions
- Parametric Study – Summary of Results

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Outline of LRFD Sections

1. Introduction
2. General Design and Location Features
3. Loads and Load Factors
4. Structural Analysis and Evaluation
5. Concrete Structures
6. Steel Structures
7. Aluminum Structures
8. Wood Structures
9. Decks and Deck Systems
10. Foundations
11. Abutments, Piers and Walls
12. Buried Structures and Tunnel Liners
13. Railings
14. Joints and Bearings

Outline of LRFD
Section 5. Concrete Structures

5.1 Scope
5.2 Definitions
5.3 Notation
5.4 Material Properties
5.5 Limit States
5.6 Design Considerations
5.7 Design for Flexural and Axial Force Effects
5.8 Shear and Torsion
5.9 Prestressing and Partial Prestressing
5.10 Details of Reinforcement

Major Changes

- Parallel Commentary
- Unified Concrete Provisions
- Shear Design
  - Modified Compression Field Theory
  - Strut-and-Tie Model
  - Interface (Horizontal) Shear
  - Partial Prestressing

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Unified Design Provisions for Reinforced and Prestressed Concrete

Motivation

- Emphasize common features
- Eliminate duplication
- Unify design procedures
- Promote the notion of "structural concrete"
- Introduce partially prestressed concrete

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Additional Major Changes

- Limit States
- Distribution Factors
- Load Factors and Combinations
- Vehicular Live Loads
- Dynamic Load Allowance (*IM*)
- Vessel Collision

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Limit States

General Form of a Limit State Function

$Q \leq R$ [LRFD Eq. 1.3.2.1-1]

where:

R = Factored resistance = ϕR_n

Q = Factored load = $\sum \eta_i \gamma_i Q_i$

η_i = Load modifier = $\eta_D \; \eta_R \; \eta_I$

γ_i = Load factor, i

Q_i = Load component, i

ϕ = Resistance factor

R_n = Nominal resistance

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Limit States

Load Modifiers

$$\eta_{\mathrm{i}} = \eta_{\mathrm{D}} \ \eta_{\mathrm{R}} \ \eta_{\mathrm{I}}$$

η_D = Factor relating to *ductility* [LRFD Art. 1.3.3]

η_R = Factor relating to *redundancy* [LRFD Art. 1.3.4]

η_I = Factor relating to *importance* [LRFD Art. 1.3.5]

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Limit States

Load Modifier for Ductility

- The structural system shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure.
- Related to structural behavior, not material behavior

For strength limit state:

$\eta_D \geq 1.05$ for nonductile components and connections

$\eta_D = \underline{1.00}$ for conventional designs

$\eta_D \geq 0.95$ for components and connections with additional ductility-enhancing measures

For all other limit states: $\eta_D = 1.00$

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Limit States

Load Modifier for Redundancy

- Multiple load path and continuous structures should be used.
- Main elements whose failure is expected to cause the collapse of the bridge shall be designated as failure-critical (non-redundant).

For strength limit state:

$\eta_R \geq 1.05$ for non-redundant members

$\eta_R =$ 1.00 for conventional levels of redundancy

$\eta_R \geq 0.95$ for exceptional levels of redundancy

For all other limit states: $\eta_R = 1.00$

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Limit States

Load Modifier for Operational Importance

- The owner may declare a bridge or any structural component and connection thereof to be of operational importance.

For strength limit state:

$\eta_I \geq 1.05$ for important bridges

$\eta_I =$ 1.00 for typical bridges

$\eta_I \geq 0.95$ for relatively less important bridges

For all other limit states: $\eta_I = 1.00$

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Limit States

Limit States for Prestressed Concrete Girders

- Strength limit state
- Service limit state
- Fatigue and fracture limit state
- Extreme event limit state

All limit states shall be considered of equal importance [LRFD Article 1.3.2.1]

Additional Note for Fatigue:

LRFD Art. 5.5.3.1 states that the fatigue limit state need **not** be checked for **fully prestressed components** designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in Table 5.9.4.2.2-1

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Limit States

Strength Limit State

- Increased vehicular live load
- Reduced load factors
- Result: Design effects are similar to Standard Specifications

Service Limit State

- Increased vehicular live load
- Similar stress limits
- Result: Design effects are more restrictive than Standard designs
- Service III added to address this situation by reducing live load effects

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Limit States

Strength limit state relates to the local and global, strength and stability.

***Strength I*:** [LRFD Art. 3.4.1]

- "Basic load combination relating to the normal vehicular use of the bridge without wind."

Maximum: $\mathbf{Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)}$

Minimum: $\mathbf{Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)}$

DC = Dead load of structural components and non-structural attachments.
DW = Dead load of wearing surface and utilities.
LL = Vehicular live load.
IM = Vehicular dynamic load allowance

Standard Specifications: $Q = 1.30D + 2.17(L+I)$

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Limit States

Service Limit States restrict the stress, deformation and crack width under regular serviceability conditions.

Service I: [LRFD Art. 3.4.1]

- "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 PC components is investigated using the following load combination

$$Q = 1.00(DC + DW) + 1.00(LL + IM)$$

Standard Specifications: $Q = 1.00\ D + 1.00(L+I)$

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Limit States

***Service III*:** [LRFD Art. 3.4.1]

- "Load combination relating only to tension in PC superstructure components with the objective of crack control."
- Tension in PC components is investigated using the following load combination

$$\boldsymbol{Q = 1.00(DC + DW) + 0.80(LL + IM)}$$

Standard Specifications: $\boldsymbol{Q = 1.00\,D + 1.00(L+I)}$

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Allowable Stress Limits

LRFD Art. 5.9.4.2.1

Stage of Loading	**Type of Stress** Compressive (Service I) Tensile (Service III)	**Allowable Stress Limits**	
		LRFD	Standard
		$\sqrt{f'_c}$ or f'_{ci} (ksi)	$\sqrt{f'_c}$ or f'_{ci} (psi)
Initial Loading Stage at Transfer	Compressive	$0.6f'_{ci}$	$0.6f'_{ci}$
	Tensile	$0.24\sqrt{f'_{ci}}$	$7.5\sqrt{f'_{ci}}$
Intermediate Loading Stage at Service	Compressive	$0.45f'_c$	$0.4f'_c$
	Tensile	$0.19\sqrt{f'_c}$	$6\sqrt{f'_c}$
Final Loading Stage at Service	Compressive	$0.6\phi_\omega f'_c$	$0.6f'_c$
	Additional Compressive Stress Check	$0.4f'_c$	$0.4f'_c$
	Tensile	$0.19\sqrt{f'_c}$	$6\sqrt{f'_c}$

Note: $0.19\sqrt{f'_c(ksi)} = 6\sqrt{f'_c(psi)}$

$0.24\sqrt{f'_{ci}(ksi)} = 7.59\sqrt{f'_{ci}(psi)}$

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Allowable Stress Limits

LRFD and Standard Specifications allow this larger tensile stress limit at transfer when additional bonded reinforcement is provided to resist the total tensile force in the concrete when the tensile stress exceeds $3\sqrt{f'_{ci}(psi)}$ or 200 psi, whichever is smaller.

ϕ_{ω} = a reduction factor to account for the fact that the unconfined concrete of the compression sides of the box girders are expected to creep to failure at a stress far lower than the nominal strength of the concrete. [LRFD Art. 5.7.4.7.1]

f'_c = concrete strength at service

f'_{ci} = concrete strength at release

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Resistance Factors

Limit State	Standard	LRFD Art. 5.5.4.2
Flexure – RC	0.90	0.90
Flexure – PC	1.00	1.00
Shear – RC	0.85	**0.90**
Shear – PC	0.90	0.90
Compression	0.70 / 0.75	**0.75**
Bearing	0.70	0.70

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HL-93 Notional Load

- ***Vehicular live load*** shall consist of a combination of:

  Design Truck **<u>OR</u>** Design Tandem

  <u>PLUS</u>

  Design Lane Load
  - <u>HL</u> stands for Highway Loading
  - <u>93</u> stands for the year introduced

- ***Each design lane*** shall be occupied by either the design truck or tandem, coincident with the lane load.

- The loads shall be assumed to occupy 10 ft. transversely within a design lane.

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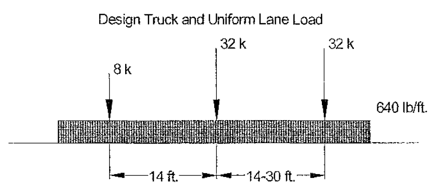
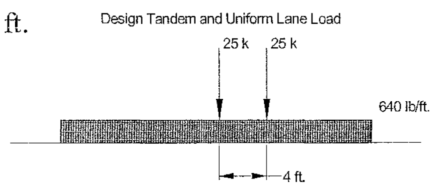
HL-93 Notional Load

- ***Design Truck***
  - Same as HS20-44
  - Three axles: 8 kips, 32 kips and 32 kips, spaced at 14 ft. and 14-30 ft.
- ***Design Tandem***
  - Pair of 25 kip axles 4 ft. apart.
  - Transverse spacing of wheels = 6 ft.
- ***Design Lane Load***
  - 0.64 kips/ft. in the longitudinal direction
  - Uniformly distributed transversely over a 10 ft. width





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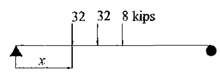
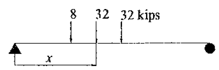
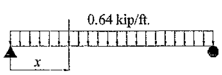
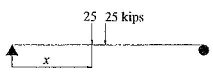
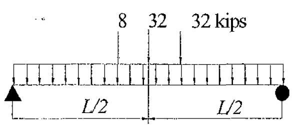
Application of HL-93 Notional Load

Live Load Placement and Formulas

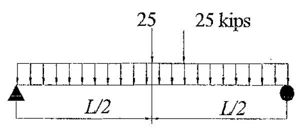
Case	Load Configuration	Moment (kips-ft) and Shear (kips) Formulas	Loading and Limitations (x and L in feet)
I		$M(x)=\frac{72(x)[(L-x)-9.33]}{L}$ $V(x)=\frac{72[(L-x)-9.33]}{L}$	Truck loading $L > 28$ ft.; for $M(x)$ $L > 14$ ft.; for $V(x)$ $x > 0$ $0 < (x/L) \leq 0.333$
II		$M(x)=\frac{72(x)[(L-x)-4.67]}{L}-112$ $V(x)=\frac{72[(L-x)-4.67]}{L}-8$	Truck loading $L > 28$ ft.; for $M(x)$ 42 ft. $\geq L > 28$ ft.; for $V(x)$ $x > 14$ ft. $0.333 < (x/L) \leq 0.5$
III		$M(x)=\frac{0.64(x)(L-x)}{2}$ $V(x)=\frac{0.64}{2L}(L-x)^2$	Lane Loading
IV		$M(x)=50(x)\left(\frac{L-x-2}{L}\right)$ $V(x)=50\left(\frac{L-x-2}{L}\right)$	For $L \leq 40$ ft., tandem loading governs in comparison to truck loading

Application of HL-93 Notional Load

Maximum Moment Calculation for Simple Span Bridges



CASE II + III



CASE IV + III

$$M_{max} = M_{truck} + M_{lane}$$

$$M_{truck} = \frac{72(x)[(L-x)-4.67]}{L} - 112$$

$$M_{lane} = \frac{0.64(x)(L-x)}{2}$$

$$M_{max} = M_{tandem} + M_{lane}$$

$$M_{tandem} = 50(x)\left(\frac{L-x-2}{L}\right)$$

$$M_{lane} = \frac{0.64(x)(L-x)}{2}$$

where, x is the distance from the left support to the section being considered

Load Distribution

LRFD allows the designer several methods of analyses and provides guidelines. [LRFD Art. 4.4 & 4.6.3]

- Refined Methods of Analysis
  - Classical Force and Displacement Methods
  - Finite Difference Method
  - Finite Element Method
  - Grillage Analogy Method
  - Miscellaneous Others
- Approximate Method
  - Distribution Factors

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Load Distribution - Approximate Method of Analysis [LRFD Art. 4.6.2.2.1]

"Bridges not meeting the requirements of LRFD Art. 4.6.2.2.1 shall be analyzed by refined analysis methods."

General Limitations:

- Width of deck is constant
- Unless otherwise specified, the number of beams ≥ 4
- Beams are parallel and have approximately the same stiffness
- Unless otherwise specified, the roadway part of the overhang, $d_e \leq 3.0$ ft.
- Curvature in plan is less than the limit specified in Art. 4.6.1.2
- Cross-section is consistent with those given in Table 4.6.2.2.1-1

The Standard Specifications do not impose such limitations.

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Load Distribution - Approximate Method of Analysis [LRFD Art. 4.6.2.2.1]

Distribution of Permanent Dead Loads:

- If all the limitations are satisfied (previous slide), the permanent dead loads can be distributed uniformly among the beams.

The Standard Specifications do not impose such a limitation.

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Types of Distribution and Correction Factors

Moment

- Section Types
- Interior Beams
- Exterior Beams
- Skewed Supports

Shear

- Section Types
- Interior Beams
- Exterior Beams
- Obtuse Corners

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Load Distribution

Variable Definitions:

S = Girder spacing, ft.

L = Span length, ft.

d = Girder depth, in.

e = Exterior girder correction factor

d_e = Distance from the exterior web of exterior girder to the interior edge of curb or traffic barrier, ft.

$g_{interior}$ = Girder distribution factor for interior girder

$g_{exterior}$ = Girder distribution factor for exterior girder

K_g = Longitudinal stiffness parameter, in.^{4}

t_s = Depth of the concrete slab, in.

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Load Distribution

LRFD Live Load Distribution Provisions for Concrete Deck on Spread Box Beams		
Category	**Distribution Factor Formulas**	**Range of Applicability**
Moment in Interior Beams	One Design Lane Loaded: $\left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12.0L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125}$	$6.0 \le S \le 18.0$ (ft.) $20 \le L \le 140$ (ft.) $18 \le d \le 65$ (in.) $N_b \ge 3$
	Use Lever Rule	$S > 18.0$ (ft.)
Moment in Exterior Beams	One Design Lane Loaded: Lever Rule Two or More Design Lanes Loaded: $g_{exterior} = e \times g_{interior}$ $e = 0.97 + \frac{d_e}{28.5}$	$0 \le d_e \le 4.5$ (ft.) $6.0 \le S \le 18.0$ (ft.)
	Use Lever Rule	$S > 18.0$ (ft.)

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Load Distribution

LRFD Live Load Distribution Provisions for Concrete Deck on Spread Box Beams		
Category	**Distribution Factor Formulas**	**Range of Applicability**
Shear in Interior Beams	*One Design Lane Loaded:* $\left(\frac{S}{10}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1}$ *Two or More Design Lanes Loaded:* $\left(\frac{S}{7.4}\right)^{0.8}\left(\frac{d}{12.0L}\right)^{0.1}$	$6.0 \le S \le 18.0$ (ft.) $20 \le L \le 140$ (ft.) $18 \le d \le 65$ (in.) $N_b \ge 3$
	Use Lever Rule	$S > 18.0$ (ft.)
Shear in Exterior Beams	*One Design Lane Loaded:* Lever Rule *Two or More Design Lanes Loaded:* $g = e \times g_{interior}$ $e = 0.8 + \frac{d_e}{10}$	$0 \le d_e \le 4.5$ (ft.)
	Use Lever Rule	$S > 18.0$ (ft.)

The Standard Specifications recommend the use of S/11 as distribution factor, where S is the girder spacing in ft.

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Load Distribution

LRFD Live Load Distribution Provisions for Concrete Deck on I-Beams		
Category	**Distribution Factor Formulas**	**Range of Applicability**
Moment in Interior Beams	*One Design Lane Loaded:* $0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ *Two or More Design Lanes Loaded:* $0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ (ft.) $4.5 \le t_s \le 12.0$ (in.) $20 \le L \le 240$ (ft.) $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
	Use lesser of the values obtained from the equation above with $N_b = 3$ or the Lever Rule	$N_b = 3$
Moment in Exterior Beams	*One Design Lane Loaded:* Lever Rule *Two or More Design Lanes Loaded:* $g_{exterior} = e \times g_{interior}$ $e = 0.77 + \frac{d_e}{9.1}$	$-1.0 \le d_e \le 5.5$ (ft.)
	Use lesser of the values obtained from the equation above with $N_b = 3$ or the Lever Rule	$N_b = 3$

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Load Distribution

LRFD Live Load Distribution Provisions for Concrete Deck on I-Beams		
Category	**Distribution Factor Formulas**	**Range of Applicability**
Shear in Interior Beams	*One Design Lane Loaded*: $0.36+\frac{S}{25.0}$ *Two or More Design Lanes Loaded*: $0.36+\frac{S}{25.0}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ (ft.) $20 \le L \le 240$ (ft.) $4.5 \le t_s \le 12.0$ (in.) $N_b \ge 4$
	Use Lever Rule	$N_b = 3$
Shear in Exterior Beams	*One Design Lane Loaded*: Lever Rule *Two or More Design Lanes Loaded*: $g = e \times g_{interior}$ $e = 0.6+\frac{d_e}{10}$	$-1.0 \le d_e \le 5.5$ (ft.)
	Use Lever Rule	$N_b = 3$

The Standard Specifications recommend the use of S/11 as distribution factor, where S is the girder spacing in ft.

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Load Distribution – Skew Correction

Reduces Load Distributed to Ext Girders.

LRFD Table 4.6.2.2.2e-1 Correction for Moment in Longitudinal Beams on Skew Supports for Spread Box Beams

Type of Superstructure	Any Number of Design Lanes Loaded	Range of Applicability
Concrete Deck on Spread Box Beams	$1.05-0.25\ \tan\theta \le 1.0$ If $\theta > 60^\circ$ use $\theta = 60^\circ$	$0^\circ \le \theta \le 60^\circ$

- Reduces the Moment Distribution Factors [LRFD Table 4.6.2.2.2e-1]
- Increases the Distribution Factors for Support Shear in Obtuse Corners [LRFD Table 4.6.2.2.3c-1]

The Standard Specifications do not take into account the affects due to skew.

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Load Distribution - Shear in Obtuse Corner

- Shear in the exterior beam of the obtuse corner should be adjusted for a skewed bridge
- [LRFD Table A4.6.2.2.3c-1]
  - For Spread Box Beams
    - Shear correction factor for skew can increase from 1.1 to 1.85
    - LRFD limits use of shear correction factor for skew to girder spacings between 6 ft. and 11.5 ft.

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Debonding Limits LRFD vs. TxDOT Practice

TxDOT Practice

- Debonding limited to 75% per row and 75% per section.
- Maximum debonding length limited to lesser of: 15 ft., 0.2 * span length, or 1/2 span length minus max. development length

LRFD [Art. 5.11.4.3]

- Debonding limited to 40% per row and 25% per section
- The use of greater percentages of partially debonded strands is allowed based on the successful past practices.

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Debonding Limits - LRFD

Additional Details [LRFD Art. 5.11.4.3]

- Debonding length of any strand shall be such that all limit states are satisfied.
- Not more than 40% of the debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any section.
- Debonded strands shall be symmetrically distributed about the centerline of the member.
- Pairs of symmetrically debonded strands should have equal debonded length.
- Exterior strands in each horizontal row shall be fully bonded.

Debonding Research

- LRFD derives its debonding limits based on a FDOT study where a specimen with 40% debonded strands (0.6 in. diameter) had inadequate shear capacity.
- Barnes, Burns and Kreger (1999) recommended that up to 75% of the strands can be debonded, if
  1. Cracking is prevented in or near the transfer length
  2. AASHTO LRFD (1998) rules for terminating the tensile reinforcement are applied to the bonded length of prestressing strands.
- Abdalla, Ramirez and Lee (1993) recommended limiting debonding to 67% per section
- In the last two studies,
  - None of the specimens failed in shear
  - All the specimen failed in pure flexure, flexure with slip, and bond failure mechanisms.

Prestress Losses

- Prestress losses for prestressed concrete members are based on a similar pattern as used in Standard Specifications.
- Divided into two categories
  - Instantaneous losses
  - Time-dependent losses
- Instantaneous losses include
  - Loss due to elastic shortening
  - Loss due to relaxation of steel at transfer

> - *Prestress loss due to relaxation of steel at transfer is not included in the Standard Specifications*
> - *TxDOT currently includes half the final relaxation loss in the instantaneous losses*

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Prestress Losses (Cont.)

- Similar expression as STD for prestress loss due to elastic shortening.
- Prestress loss due to relaxation of steel at transfer for members with low-relaxation strands is given as

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right]f_{pj} \quad \text{[LRFD Eq. 5.9.5.4.4b-2]}$$

f_{pu} = Ultimate stress in prestressing steel
f_{pj} = Initial stress in tendon at the end of stressing
t = Time estimated in days from stressing to transfer
f_{py} = Yield strength of prestressing steel

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Prestress Losses (Cont.)

- Two options for estimation of time-dependent losses are provided
  - Approximate Lump-Sum Estimate
  - Refined Estimates
- Approximate Lump-sum losses for pretensioned members stressed after attaining a compressive strength of 3.5 ksi are applicable if:
  - Normal weight concrete is used
  - Concrete is steamed or moist cured
  - Normal or low-relaxation prestressing strands or bars are used
  - Average exposure conditions and temperatures exist at the site.

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Prestress Losses (Cont.)

- LRFD Table 5.9.5.3.1 provides the lump-sum time dependent losses

Table 5.9.5.3-1 Time-Dependent Losses in ksi.

Type of Beam Section	Level	For Wires and Strands with f_{pu} = 235, 250 or 270 ksi	For Bars with f_{pu} = 145 or 160 ksi
Rectangular Beams, Solid Slab	Upper Bound Average	$29.0 + 4.0\,PPR$ $26.0 + 4.0\,PPR$	$19.0 + 6.0\,PPR$
Box Girder	Upper Bound Average	$21.0 + 4.0\,PPR$ $19.0 + 4.0\,PPR$	15.0
I-Girder	Average	$33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right] + 6.0\,PPR$	$19.0 + 6.0\,PPR$
Single T, Double T, Hollow Core and Voided Slab	Upper Bound Average	$39.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right] + 6.0\,PPR$ $33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right] + 6.0\,PPR$	$31.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right] + 6.0\,PPR$

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Prestress Losses

- Refined estimates yield more accurate results as compared to Lump-sum method.
- Refined estimates for prestressed concrete members provided by LRFD are applicable if:
  - Span is not greater than 250 ft.
  - Normal weight concrete is used
  - Compressive strength of concrete is in excess of 3.5 ksi at the time of prestress.
- Refined estimate of prestress losses in pretensioned members with low-relaxation strands includes
  - Loss due to steel relxation after transfer (similar expression as STD)
  - Loss due to concrete shrinkage (similar expression as STD)
  - Loss due to concrete creep (similar expression as STD)

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Shear Design by Modified Compression Field Theory (MCFT)

- Modified compression field theory
  - unified method, applicable to prestressed and nonprestressed concrete members
  - based on equilibrium, compatibility and stress-strain relationships
  - is a rational method, showing the significance of the parameters involved
  - based on variable angle truss analogy (as compared to the constant 45° truss analogy used by traditional theories)
  - accounts for the tension in the longitudinal reinforcement due to shear and the stress transfer across the cracks

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Shear Design by Modified Compression Field Theory (MCFT)

- takes into account the shear stress and strain conditions at the section
- shear strength of concrete is determined using a factor β, which indicates the ability of diagonally cracked concrete to transfer tension
- the angle of inclination of diagonal compressive stress, θ is used to determine the critical section for shear
- if θ = 45° and β = 2 is used, this theory yields same results as 45° truss analogy.

Shear Design by Modified Compression Field Theory (MCFT)

- LRFD Specifications provides an extensive commentary and the mechanics of the MCFT.
- Entirely different design approach as compared to the Standard Specifications.
- In STD the shear strength of concrete is calculated as the lesser of
  - nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, V_{ci}
  - Nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in the web, V_{cw}

Shear Design by Modified Compression Field Theory (MCFT)

- The LRFD shear design procedure involves the following steps

$$V_n = V_c + V_s + V_p$$

$$V_n = 0.25 f_c' b_v d_v$$

V_n = Nominal shear resistance, kips

V_c = Concrete contribution = $0.0316\,\beta\,\sqrt{f'_c}\,b_v\,d_v$

V_p = Vertical component of prestressing steel, kips

V_s = Transverse reinforcement contribution

$$= \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$

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Shear Design by Modified Compression Field Theory (MCFT)

A_v = Area of shear reinforcement within a distance s, in.^{2}

s = Spacing of stirrups, in.

f_y = Yield strength of shear reinforcement, ksi

α = Angle of inclination of transverse reinforcement to the longitudinal axis

b_v = Effective web width taken as the minimum web width within the depth d_v, in.

d_v = Effective shear depth, in.

θ = Angle of inclination of diagonal compressive stresses

β = Factor indicating the ability of diagonally cracked concrete to transmit tension.

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Shear Design by Modified Compression Field Theory (MCFT)

- Critical section for shear
  - The critical section for shear shall be taken as greater of d_v or $0.5d_v \cot\theta$
  - The critical section calculation is a iterative process as θ is unknown at the beginning of the design.
  - θ is assumed (around 23° is a good assumption) and is updated if needed based on the results.

The critical section of shear is given as $h_c/2$ for Standard specifications.

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Shear Design by Modified Compression Field Theory (MCFT)

- Determination of θ and β
  - The values are interpolated from the Tables provided in the LRFD using the shear stress and strain values for the section.
- The shear stress in concrete is given as

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$

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Shear Design by Modified Compression Field Theory (MCFT)

- Longitudinal Strain in the concrete is calculated as

Case 1: At least minimum transverse reinforcement is provided

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_s A_s + E_p A_{ps})} \leq 0.001$$

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Shear Design by Modified Compression Field Theory (MCFT)

Case 2: Less than minimum transverse reinforcement is provided

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{(E_s A_s + E_p A_{ps})} \leq 0.002$$

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Shear Design by Modified Compression Field Theory (MCFT)

Case 3: If the strain is found to be negative from the two equations presented

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps} f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

Determination of θ and β

Table 5.8.3.4.2-1 Values of θ and β for Sections with Transverse Reinforcement.

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

Shear Design by Modified Compression Field Theory (MCFT)

- Longitudinal Reinforcement Requirement
  - LRFD specifies that at each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be such that

$$A_s f_y + A_{ps} f_{ps} \geq \frac{M_u}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5 V_s - V_p \right) \cot\theta$$

There was no such requirement for Standard designs

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Interface Shear Design Provisions

- Significant changes as compared to Standard Specifications.
- Formulas based on shear-friction theory replaced the empirical formulas used in Standard Specifications.
- LRFD Procedure:

  Step 1: Compute required horizontal shear per unit length of girder

$$V_h = \frac{V_u}{d_e}$$

V_u = Factored vertical shear due to superimposed and live loads, kips

d_e = Distance between the centroid of steel in the tension side of the beam to the center of the compression block (center of the deck can be used for simplicity)

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Interface Shear Design Provisions

Step 2: Calculate the nominal shear resistance at the section

$$V_n = cA_{cv} + \mu[A_{vf}f_y + P_c]$$

Note that V_h has units of kip/in., hence V_n is calculated on a per in. basis for consistency of units.

A_{cv} = Area of concrete engaged in shear transfer, in.^{2} (taken on a per in. basis as b_v*1 in., where b_v is the width of interface)

A_{vf} = area of shear reinforcement crossing the shear plane, in.^{2}

f_y = Yield strength of reinforcement, ksi

P_c = Permanent net compressive force normal to the shear plane, kips

c = Cohesion factor

μ = Friction factor

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Interface Shear Design Provisions

For concrete placed against clean, hardened concrete with surface intentionally roughened to an amplitude of 0.25 in.

c = **0.100 ksi**

μ = **1.0** for normal weight concrete

For concrete placed against hardened concrete clean and free of laitance, but not intentionally roughened.

c = **0.075 ksi**

μ = **0.6** for normal weight concrete

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Interface Shear Design Provisions

- Solve for A_{vf} such that

$$V_h = \phi V_n$$

- Check for nominal shear resistance

$$V_n \leq 0.2 f_c' A_{cv}$$
$$V_n \leq 0.8 A_{cv}$$

- Check for minimum reinforcement area

$$A_{vf} \geq \frac{0.05\, b_v}{f_y}$$

b_v = width of interface, in.

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Interface Shear Design Provisions

Standard Specifications Provisions

- The shear strength of the section is chosen based on the following cases:
  - $V_n < 80\, b_v d$, if the surface is clean and intentionally roughened
  - $V_n < 80\, b_v d$, if minimum ties are provided and surface in not roughened
  - $V_n < 350\, b_v d$, if surface is roughened to approximately ¼ in. and minimum ties are provided (This case almost always governs)
  - V_n may by increased by $(160 f_y/40{,}000) b_v d$ for each percent of tie reinforcement provided in excess of minimum reinforcement.

- Minimum reinforcement area is given as $50 b_v s / f_y$

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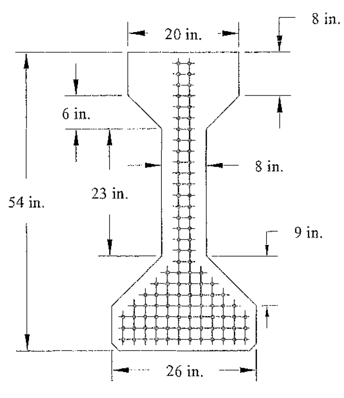
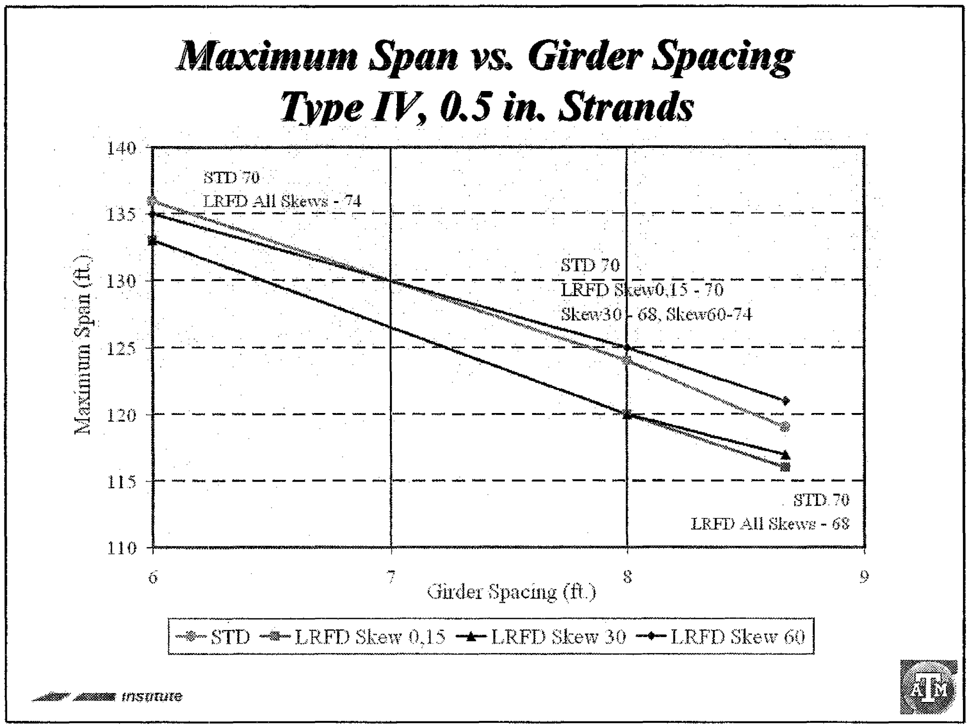
Parametric Study

Parameter	Description / Selected Values
Design Codes	AASHTO Standard Specifications, 17th Ed. (2002) AASHTO LRFD Specifications, 3rd Ed. (2004)
Girder Section	Type C, Type IV and U54
Girder Spacing	Type C: 6'-0", 8'-0" and 8'-8" Type IV: 6'-0", 8'-0" and 8'-8" U54: 8'-6", 10'-0", 11'-6", 14'-0" and 16'-8"
Spans	40 ft. to max. span at 10 ft. intervals for Type C beams 90 ft. to max. span at 10 ft. intervals for Type IV and U54 beams
Strand Diameter	0.5 in. and 0.6 in.
f'_{ci}	varied from 4000 to 6750 psi
f'_c	varied from 5000 to 8500 psi (up to 8750 psi for optimization on longer spans)
Skew Angle	0, 15, 30 and 60 degrees

Sample Parametric Study Results Type IV and Type C Girders



AASHTO Type IV Girder Section


Maximum Span vs. Girder Spacing
Type IV, 0.5 in. Strands
STD 70
LRFD All Skews - 74
STD 70
LRFD Skew0,15 - 70
Skew30 - 68, Skew60-74
STD 70
LRFD All Skews - 68
Maximum Span (ft.)
Girder Spacing (ft.)
STD
LRFD Skew 0,15
LRFD Skew 30
LRFD Skew 60
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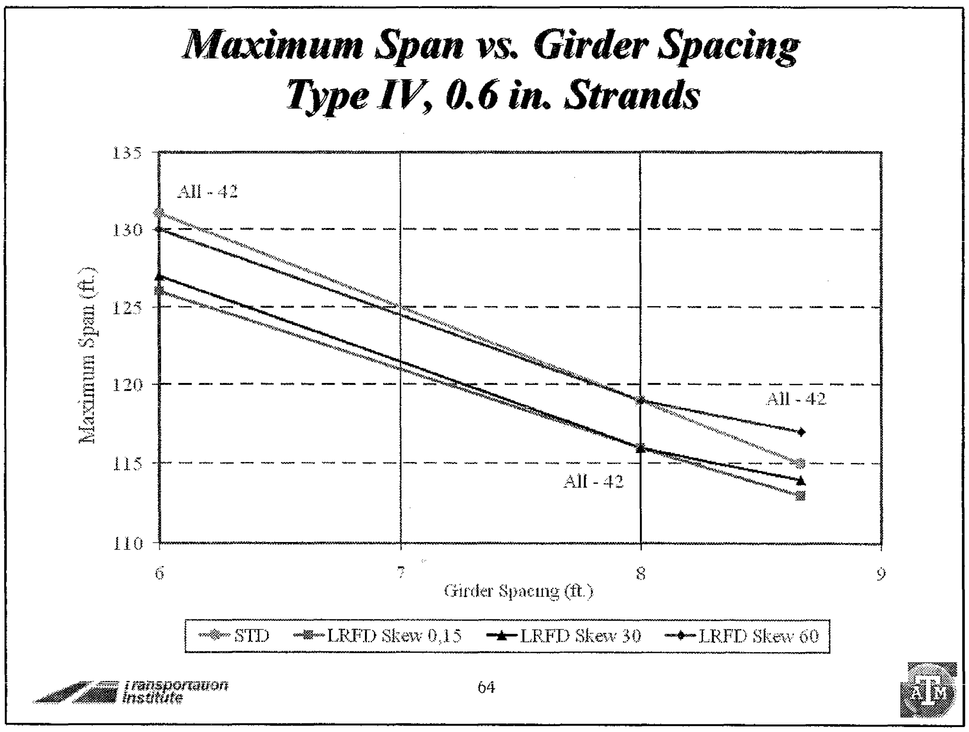
Maximum Span vs. Girder Spacing
Type IV, 0.6 in. Strands
All - 42
All - 42
All - 42
Maximum Span (ft.)
Girder Spacing (ft.)
STD
LRFD Skew 0,15
LRFD Skew 30
LRFD Skew 60
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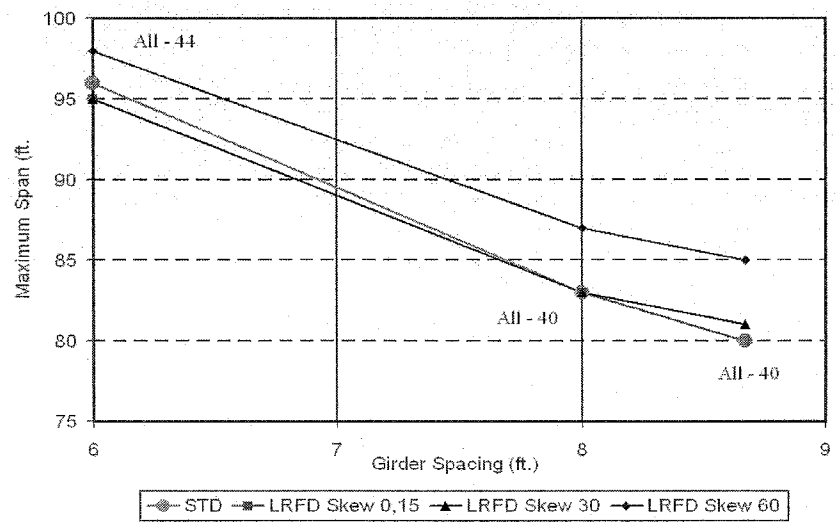
Maximum Span vs. Girder Spacing Tvpe C. 0.5 in. Strands



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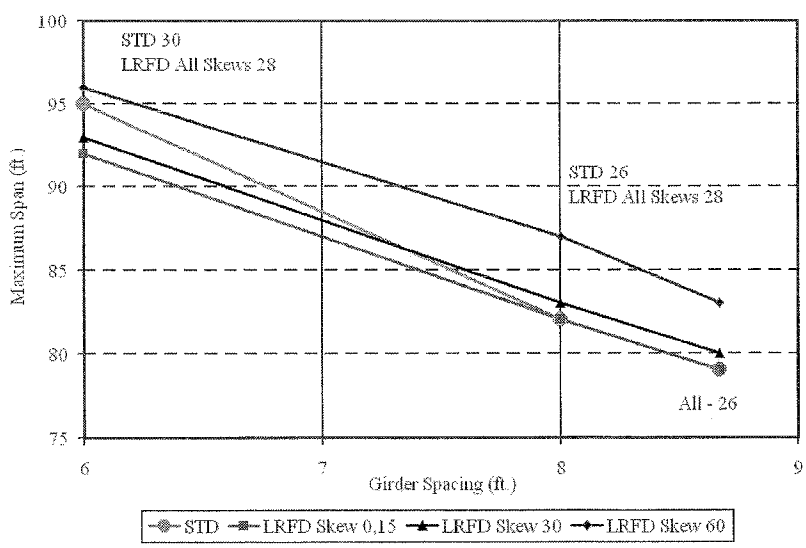
Maximum Span vs. Girder Spacing Type C, 0.6 in. Strands



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Max. Differences in Maximum Span Length LRFD vs. Standard Designs (Type IV Girders)

Girder Spacing (ft.)	Strand Dia. = 0.5 in.		Strand Dia. = 0.6 in.	
	Difference	% Diff.	Difference	% Diff.
6	-3.0 ft.	-2.21	-5.0 ft.	-3.82
8	-4.0 ft.	-3.23	-3.0 ft.	-2.52
8.67	-3.0 ft.	-2.52	2.0 ft.	1.74

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Max. Differences in Maximum Span Length LRFD vs. Standard Designs (Type C Girders)

Girder Spacing (ft.)	Strand Dia. = 0.5 in.		Strand Dia. = 0.6 in.	
	Difference	% Diff.	Difference	% Diff.
6	3.0 ft.	3.17	4.0 ft.	4.17
8	4.0 ft.	4.60	5.0 ft.	6.00
8.67	5.0 ft.	6.00	4.0 ft.	5.06

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Observations for Type IV Girders (LRFD vs. Std.)
Live Load Moment

- Undistributed midspan LL moments *increased* 48-56%
- Moment DFs *decreased* 2-30%
  - LRFD yields smaller DFMs for all spans, girder spacing and skew angles.
  - Difference increases with an increase in skew angle, span length or girder spacing
- Distributed midspan (LL+I) moments *increased* 4-52%
  - LRFD yields greater moments for all spans, girder spacing and skew angles. The difference is
    - Decreasing with increase in skew angle or girder spacing
    - Increasing with increase in span length

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Observations for Type IV Girders (LRFD vs. Std.)
Live Load Shear

- Undistributed LL shears at critical section *increased* 35-54%
- Shear Distribution Factors *increased* 9-23%
  - LRFD yields larger DFVs for all spans, girder spacing and skew angles.
  - The difference is decreasing with increase in girder spacing
- Distributed (LL+I) shears at critical section *increased* 56-99%
  - LRFD yields significantly greater shears for all spans, girder spacing and skew angles. The difference is
    - Increasing with increase in span length
    - Decreasing with increase in girder spacing

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Observations for Type IV Girders (LRFD vs. Std.)
Impact Load

- For LRFD: constant at 33% of live load
- For Standard: varies from 19 - 23% of live load

Observations for Type IV Girders (LRFD vs. Std.)
Service Load Design

- Initial prestress loss *increased*
  - 2-17% for 0.5 in. diameter strands
  - 0-16% for 0.6 in. diameter strands
- Final prestress loss *increased*
  - 1-17% for 0.5 in. diameter strands
  - 1-20% for 0.6 in. diameter strands
- Change in prestress loss due to elastic shortening *varies*
  - -5-14% for 0.5 in. diameter strands
  - -5-12% for 0.6 in. diameter strands

Observations for Type IV Girders (LRFD vs. Std.)
<u>Service Load Design</u>

- Prestress loss due to concrete <u>shrinkage</u> – *No effect*
- Change in prestress loss due to <u>creep</u> of concrete *varies*
  - -13-15% for 0.5 in. diameter strands
  - -2-30% for 0.6 in. diameter strands
- Prestress loss due to <u>initial steel relaxation</u> (using ½ final steel relaxation as initial steel relaxation for STD) *increased*
  - 36-223% for 0.5 in. diameter strands
  - 48-168% for 0.6 in. diameter strands
- Prestress loss due to <u>total steel relaxation</u> *increased*
  - 78-168% for 0.5 in. diameter strands
  - 94-154% for 0.6 in. diameter strands

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Observations for Type IV Girders (LRFD vs. Std.)
<u>Service Load Design</u>

- <u>Number of Strands</u>
  - LRFD Designs: change is -4 to 8 strands (-6% to 13%)
    - Skew angles less than 30°: up to 8 more strands
    - Skew angle = 60°: up to 4 fewer strands
  - Trend explained
    - Distributed Live Load Moment increased significantly except for 60° skew angles, causing larger bottom tensile stresses
    - Increased prestress losses

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Observations for Type IV Girders (LRFD vs. Std.) <u>*Service Load Design*</u>

- <u>Concrete Strength</u>
  - Required concrete strength at <u>release</u> varies: -6 to 13%
    - Trend explained:
      - Increase in number of strands
      - Increased prestress force causing larger initial stresses at girder ends.
  - Required concrete strength at <u>service</u> varies: -9 to 7%
    - Trend explained:
      - Difference is 0 for most of the cases (5,000 psi governs)
      - For few cases the increase in number of strands causes an increase in required concrete strength
      - Few cases are governed by the concrete strength at release.

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Observations for Type IV Girders (LRFD vs. Std.) <u>*Service Load Design*</u>

- <u>Span Length</u>
  - *Smaller* span lengths are possible with LRFD designs (up to 4% decrease) for all skew angles except 60°.
  - *Longer* span lengths for 60° skew (up to 2% increase)
  - Trends Explained
    - Increase in the distributed live load moments
    - Larger number of strands, requiring larger concrete strengths due to increased prestress.
    - Increase in initial and final prestress losses
    - Live load moments are smaller for 60° skew, resulting in a slight increase in maximum span lengths

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Observations for Type IV Girders (LRFD vs. Std.) Flexural Strength Design

- Flexural Strength Design
  - Skew Angles less than 30°: M_u increased 1-8%
  - Skew Angle = 60°: M_u decreased 1-10%
  - Trends Explained
    - Decrease in the dead load and live load factors
    - Small differences show LRFD is calibrated to Standard Specifications for strength limit state
- Nominal Moment resistance, M_n
  - Skew Angles less than 30°: M_n increased 1-7%
  - Skew Angle = 60°: M_n decreased 1-6%
  - Trends Explained
    - Increase in number of strands and concrete strength

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Observations for Type IV Girders (cont.)

- Transverse Shear Design
  - Major differences observed in required transverse shear reinforcement area (A_v) using the MCFT (LRFD)
    - Type IV: A_v varies -33% to 203%
  - Trends Explained:
    - Critical section distance from the girder end is increased resulting in decreased shears, but
    - Method for the calculation of shear strength provided by concrete V_c is also changed, resulting in mixed trends.

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Observations for Type IV Girders (cont.)

- Interface Shear Design
  - Interface shear reinforcement area *increased* **165 to 300%**
  - Shear reinforcement *governed* by interface shear design in many cases.
    - Calculations based on shear friction theory yield a very large interface shear reinforcement area.

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Observations for Type C Girders

- Trends were similar to those for Type IV Girders

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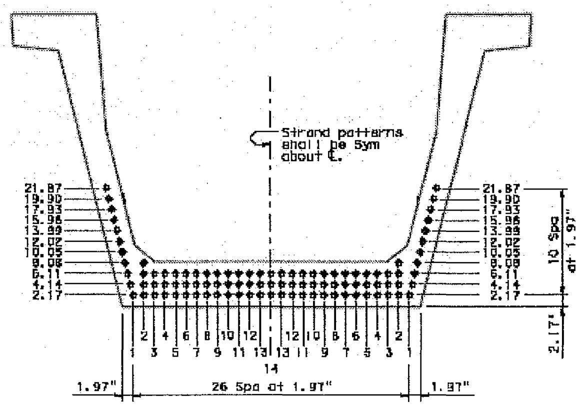
Sample Parametric Study Results
U54 Girders



Texas U54 Girder Section

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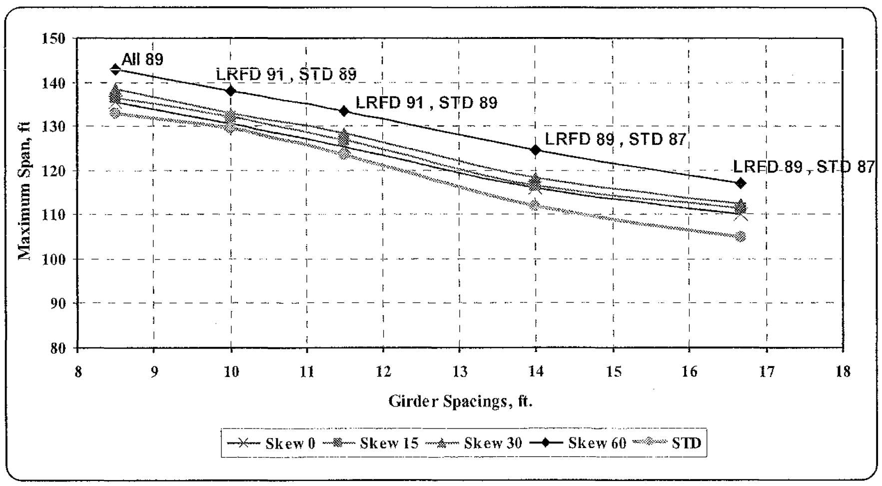
Maximum Span vs. Girder Spacing
U54, 0.5 in. Strands



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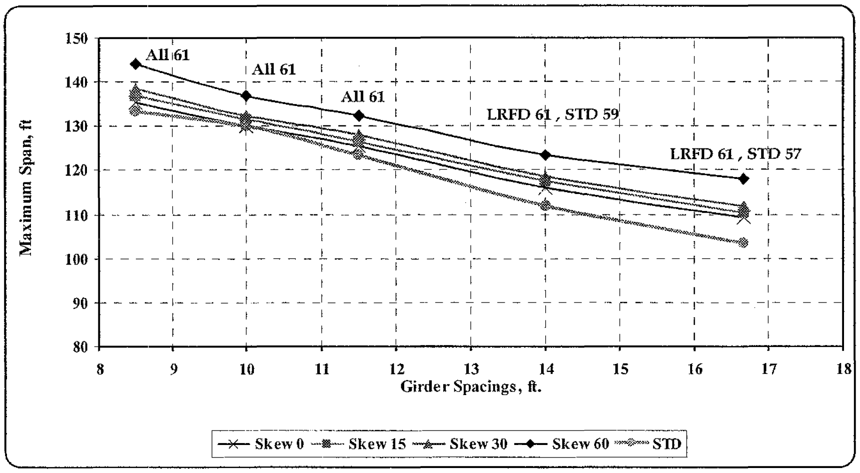
Maximum Span vs. Girder Spacing
U54, 0.6 in. Strands



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Max. Differences in Maximum Span Length
LRFD vs. STD Designs (U54 Girders)

Girder Spacing (ft.)	Strand Diameter = 0.5 in. Skew (degrees) 0, 15, 30	Strand Diameter = 0.5 in. Skew (degrees) 60	Strand Diameter = 0.6 in. Skew (degrees) 0, 15, 30	Strand Diameter = 0.6 in. Skew (degrees) 60
8.5	2.5 ft. to 4 ft. *(2% to 4.5%)*	8 ft. *(10%)*	1 ft. to 4 ft. *(2% to 5%)*	8 ft. *(10.5%)*
10	1 ft. to 3 ft. *(1% to 3.5%)*	7 ft. *(8.5%)*	0 ft. to 2 ft. *(0% to 2.5%)*	5 ft. *(7%)*
11.5	2 ft. to 4 ft. *(2% to 5%)*	8 ft. *(10%)*	2 ft. to 4 ft. *(2% to 4.5%)*	7 ft. *(9%)*
14	4 ft. to 6 ft. *(4% to 6.5%)*	11 ft. *(12.5%)*	4 ft. to 6 ft. *(4% to 6.5%)*	10 ft. *(11.5%)*
16.67	5 ft. to 7 ft. *(5% to 7.5%)*	11 ft. *(12%)*	6 ft. to 8 ft. *(6% to 8.5%)*	14 ft. *(14.5%)*

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Observations for U54 Girders (LRFD vs. STD)
Live Load Moments

Trends

- Undistributed midspan LL moments *increased* 48-71%
- Moment DFs *decreased* 23-63%
- Distributed midspan LL moments changed -40% to +16%

Distributed LL Moments

- LRFD values are higher
  - For all spacings (except 16.67 ft.) with 0° and 15° skew
- LRFD values are lower
  - For all spacings with 30° and 60° skew (except 10 ft. with 30° skew)
  - Difference increased with an increase in skew angle.

Observations for U54 Girders (LRFD vs. STD)
Live Load Shears

Trends

- Undistributed LL shears at critical section *increased* 35-56%.
- Shear DFs changed -12% to +1%.
- Distributed LL shears at critical section *increased* 25-56%

Distributed LL Shears

- LRFD values are higher
  - Difference increased with increase in girder spacing.
  - Skew angle had a negligibly small effect.

Observations for U54 Girders (LRFD vs. STD)
Dynamic Load Allowance

- For LRFD: constant at 33% of live load
- For Standard: varies from 19-23% of live load

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Observations for U54 Girders (LRFD vs. STD)
Service Load Design

- Span Length
  - LRFD designs resulted in *longer* span lengths (up to 15% increase)
    - longer with higher skew angles.
  - Longer spans are explained
    - For 30° and 60° skew
      - Significant reduction in the distributed live load moment
      - Reduction in initial and final prestress losses calculation
  - For example, for the 60° skew (span length increased up to 15%)
    - The distributed live load moment decreased up to 40.2%
    - The initial prestress losses decreased up to 19.4%
    - The final prestress losses decreased up to 17.9%

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Observations for U54 Girders (LRFD vs. STD)
Service Load Design

- Number of Strands
  - LRFD Designs: 1 to 18 fewer strands
    - Skew angles less than 30°: up to 10 fewer strands
    - Skew angle = 60°: up to 18 fewer strands
  - Trend explained
    - Distributed Live Load Moment decreased significantly for 30° and 60° skew angles.
    - The effect of live load reduction factor
      - LRFD Service III Limit State: 0.8
      - Standard Service Limit State: 1.0

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Observations for U54 Girders (LRFD vs. STD)
Service Load Design

- Concrete Strength
  - Required concrete strength at release decreased up to 25%
    - Trend explained:
      - Decrease in initial prestress losses and number of strands
      - Tensile stress limit increased from $7.5\sqrt{f'_{ci}}$ to $7.59\sqrt{f'_{ci}}$
  - Required concrete strength at service decreased up to 11%
    - Trend explained:
      - Compressive stress limit due to sustained loads increased from $0.4f'_c$ to $0.45f'_c$

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Observations for U54 Girders (LRFD vs. STD)
Service Load Design

- Prestress Losses
  - Initial prestress loss changed -23% to +8 %
  - Final prestress loss changed -18% to +7 %

  - Trend explained:
    - Initial relaxation loss decreased up to 192%
    - Final relaxation loss decreased up to 216%
    - Elastic shortening loss ranged from -7 to 31%
    - Creep loss range from -2 to 47%

- Camber: changed -45% to +6%

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Observations for U54 Girders (LRFD vs. STD)
Flexural Strength Design

- Factored Design Moment, M_u
  - Skew Angles less than 30°: M_u decreased 4-17%
  - Skew Angle = 60°: M_u decreased 19-29%

  - LRFD *values are lower*
    - Difference increased with increase in *Skew* and *Girder Spacing*

- Reduced Nominal Moment Strength, ϕM_n
  - ϕM_n decreased 3-23%
  - Because of decrease in the number of strands in LRFD designs

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Observations for U54 Girders (LRFD vs. STD)
Shear Design

- Transverse Shear Design
  - Major differences observed in required transverse shear reinforcement area (A_v) using the MCFT
  - A_v *decreased* from 35 - 49%
- Interface Shear Design
  - Interface shear reinforcement area *increased* 148 - 370%.
  - Shear reinforcement **governed** by interface shear design.

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Detailed Design Example for Interior AASHTO Type IV Prestressed Concrete Bridge Girder

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TxDOT LRFD Seminar
August 29, 2005 – College Station, Texas

Outline

Part I

- Design Parameters
- Material and Section Properties
- Loads, Moments and Shears
- Distribution of Live Load Effects (DFs)
- Summary of Changes

Part II

- Service Load Limit State Design
  - o Initial Strand Estimate
  - o Prestress Losses
  - o Final Strand Estimate
  - o Final Concrete Strength Estimate
- Summary of Changes

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Outline (cont.)

Part III

- Fatigue Limit State Design
- Flexural Strength Design
- Summary of Changes

Part IV

- Shear Design
  - Transverse Shear Design
  - Interface Shear Design
- Summary of Changes

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Outline (cont.)

Part V

- Camber and Deflections
- Comparison with Standard Specification Results
- Summary of Changes

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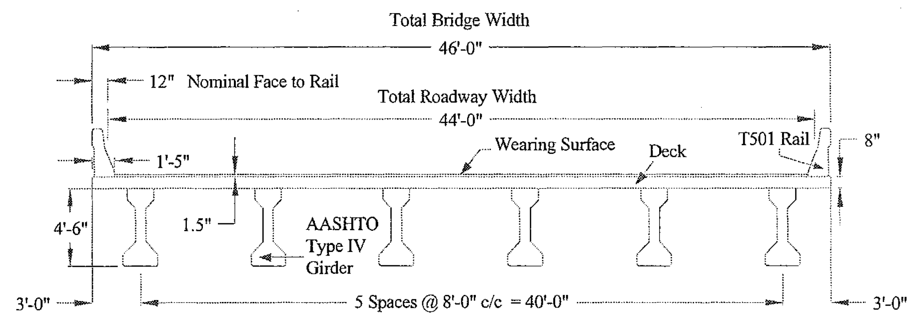
Part I

- Design Parameters
- Material and Section Properties
- Loads, Moments and Shears
- Distribution of Live Load Effects (DFs)
- Summary of Changes

Design Parameters

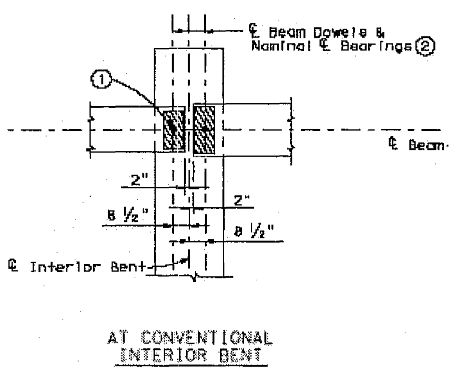
- Simple span bridge – 110 ft. c/c pier distance
- AASHTO Type IV girder spacing – 8 ft. c/c
- Total bridge width – 46'-0"
- Total roadway width – 44'-0"
- T501 type rails
- Relative humidity (RH) = 60%
- Skew angle – 0 degrees
- *AASHTO LRFD Specifications, 3rd Edition, 2004*

Design Parameters (cont.)



Bridge Cross-Section

Design Parameters (cont.)



Girder End Details

Span Length (c/c piers) = 110'-0"

Overall girder length
= 110'-0"– 2(2") = 109'-8"

Design Span (c/c of bearing)
= 110'-0"– 2(8.5") = 108'-7"

Material Properties

- Cast in place (CIP) slab (composite action)
  - o Thickness, $t_s = 8.0$ in.
  - o Concrete Strength at 28-days, $f'_c = 4.0$ ksi
  - o Thickness of asphalt wearing surface, $t_w = 1.5$ in.
  - o Unit weight of concrete, $w_c = 0.150$ kcf

- Precast girders: AASHTO Type IV
  - o Concrete Strength at release, $f'_{ci} = 4.0$ ksi *
  - o Concrete Strength at release, $f'_c = 5.0$ ksi *

  * These values are taken as an initial estimate and will be finalized based on an optimum design.

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Material Properties (cont.)

- Pretensioning strands: ½ in. diameter, seven-wire low relaxation strands
  - o Area of one strand = 0.153 in.^{2}
  - o Ultimate stress, $f_{pu} = 270$ ksi
  - o Yield strength, $f_{py} = 0.9 f_{pu} = 243$ ksi [LRFD Table 5.4.4.1-1]
  - o Stress limits for prestressing strands: [LRFD Table 5.9.3-1]
    Before transfer, $f_{pi} = 0.75 f_{pu} = 202.5$ ksi
    At service limit state (after all losses)
    $f_{pe} \leq 0.80 f_{py} = 194.4$ ksi
    (This limit is not specified by Standard Specifications)
  - o Modulus of Elasticity, $E_p = 28{,}500$ ksi [LRFD Art. 5.4.4.2]
    (This value is specified as 28,000 ksi for Standard Specifications)

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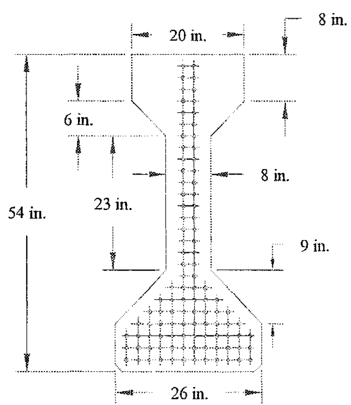
Material Properties (cont.)

- Nonprestressed reinforcement:
  - o Yield strength, f_y = 60,000 psi
  - o Modulus of Elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]
- Unit weight of asphalt wearing surface = 140 pcf
- T501 type barrier weight = 326 plf /side

Section Properties



Section Geometry and Strand Pattern for AASHTO Type IV Girder

- Depth of the section, h = 54 in.
- Area of cross section, A = 788.4 in.^{2}
- Moment of Inertia, I = 260,403 in.^{4}
- Distance from centroid to extreme top fiber, y_t = 29.25 in.
- Distance from centroid to extreme bottom fiber, y_b = 24.75 in.
- Section modulus referenced to extreme top fiber, S_t = 8,902.67 in.^{2}
- Section modulus referenced to extreme bottom fiber, S_b = 10,521.33 in.^{2}

Composite Section Properties

- Effective flange width is lesser of: [LRFD Art. 4.6.2.6.1]
  - o ¼ design span length = 325.75 in.
  - o 12*(Effective slab thickness) + [greater of web thickness (8 in.) or ½ beam top flange width (20 in.)]: 12(8) + ½ (20) = 106 in.
  - o Average spacing of adjacent girders: 8*(12 in./ft.) = 96 in. (controls)

    This is a slightly different method as compared to the Standard Specifications where the effective web width is computed, based on which the effective flange width is determined.
- Modular ratio between the slab and girder concrete, n, is taken as 1 for the service load calculations following TxDOT practice.
- In this example, the modular ratio is computed based on the actual concrete strength for use in the flexural strength, shear, and deflection calculations.

Composite Section Properties (cont.)

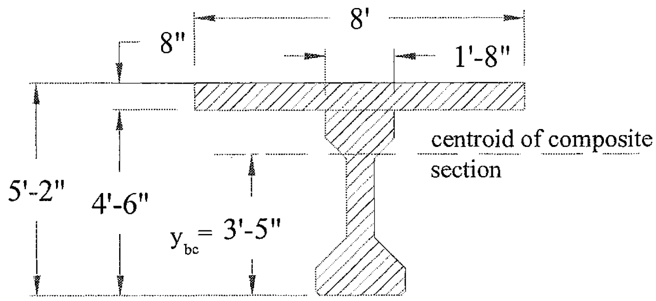
- Transformed flange width = n*(effective flange width)

  = 1*(96) = 96 in.
- Transformed flange area = n*(effective flange width) (t_s)

  = (1)(96) (8) = 768.0 in.^{2}

	Transformed Area A (in.^{2})	**y_b (in.)**	**Ay_b (in.^{3})**	**$A(y_{bc} - y_b)^2$ (in.^{4})**	**I (in.^{4})**	**$I + A(y_{bc} - y_b)^2$ (in.^{4})**
Girder	788.4	24.75	19,512.9	212,231.5	260403	472,634.5
Slab	768.0	58.00	44,544.0	217,868.9	4096	221,964.9
$\sum$	**1556.4**		64,056.9			**694,599.5**

Composite Section Properties (cont.)



Composite Section Details

Composite Section Properties (cont.)

Height of composite section	h_c = 62 in
Area of composite section	A_c = 1556.4 in.^{2}
Moment of inertia of composite section	I_c = 694,599.5 in.^{4}
Distance from centroid of composite section to extreme bottom fiber of girder	y_{bc} = 41.16 in.
Distance from centroid of composite section to extreme top fiber of girder	y_{tg} = 12.84 in.
Distance from centroid of composite section to extreme top fiber of slab	y_{tc} = 20.84 in.
Section modulus ref. to extreme bottom fiber of girder	S_{bc} = 16,876.8 in.^{3}
Section modulus ref. to extreme top fiber of girder	S_{tg} = 54,083.9 in.^{3}
Section modulus ref. to extreme top fiber of slab	S_{tc} = 33,325.3 in.^{3}

Loads

- Non-Composite Dead Loads
- Composite Dead Loads
- Live Load
- Dynamic (Impact) Load

Non-Composite Dead Loads

- Girder self weight, $w_g = 0.821$ kips/ft.
- Slab weight

$$w_S = (0.150\ \text{kips/ft.}^3)(8\ \text{ft.})\left(\frac{8\ \text{in.}}{12\ \text{in./ft.}}\right) = 0.8\ \text{kips/ft.}$$

Composite Dead Loads

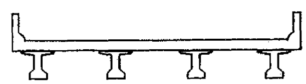
- Permanent Loads

  The permanent loads on the bridge including loads from railing and wearing surface can be distributed uniformly among all beams if the following conditions are met [LRFD Art. 4.6.2.2.1].

  (This check is not required by Standard Specifications.)

  - o Width of deck is constant (O.K.)
  - o Number of beams, N_b, is not less than four [$N_b = 6$] (O.K.)
  - o Beams are parallel and have approximately the same stiffness (O.K.)
  - o The roadway part of the overhang, $d_e \leq 3.0$ ft. [d_e = 1'-8"] (O.K.)
  - o Curvature in plan is less than 4° [Curvature = 0°] (O.K.)
  - o Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.2.1-1 [Type k] (O.K.)

composite Dead Loads (Cont...)

Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	 (k)

Type k Girder, AASHTO LRFD Specifications

Composite Dead Loads (cont.)

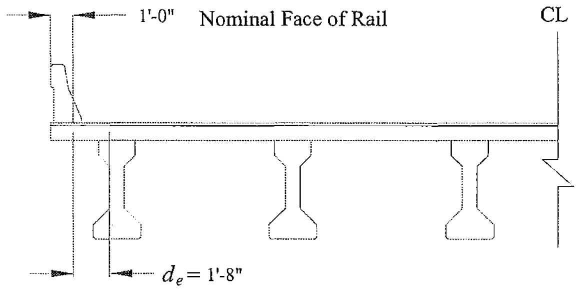


Illustration for Computation of d_e

d_e = Distance from exterior web of exterior girder to the interior edge of traffic barrier

Composite Dead Loads (cont.)

- Weight of T501 rails on each girder

$$w_{barr} = 2\left(\frac{0.326 \text{ kips/ft.}}{6 \text{ girders}}\right) = 0.109 \text{ kips/ft.}$$

- Weight of wearing surface distributed to each girder

$$w_{ws} = \frac{(0.140 \text{ kips/ft.}^3)\left(\frac{1.5 \text{ in.}}{12 \text{ in./ft.}}\right)(44 \text{ ft.})}{6 \text{ girders}} = 0.128 \text{ kips/ft.}$$

Critical Section for Shear

The calculation for the critical section for shear in LRFD design is based on an iterative process. As an initial guess the critical section is taken as

$(h_c/2)$ + (1/2 bearing width) = (62/2) + (7/2) = 34.5 in. = 2.88 ft. from the centerline of bearing.

The Standard Specifications specify the critical section for shear to be taken as a distance $h_c/2$ which is 2.58 ft. from the face of the support.

Hold Down Point

- The TxDOT *Bridge Design Manual* recommends the hold down point for harped strands to be computed as follows.

  Distance of hold down point from the midspan is greater of
  - 0.05 (span length) = 0.05 (108'-7") = 5.43 ft. *(controls)*
  - 5 ft.

  Distance of hold down point from centerline of bearing is

  0.5 (108'-7") – 5.43 ft. = 48.86 ft.

Dead Load Moments

Distance from the bearing centerline, x (ft.)	Moments at distance x from bearing centerline			
	Loading			
	Girder self weight, M_g (k-ft.)	Slab weight, M_S (k-ft.)	Barrier weight, M_{barr} (k-ft.)	Wearing surface weight, M_{ws} (k-ft.)
2.88	124.76	121.56	16.56	19.45
10.86	435.58	424.44	57.83	67.91
21.72	774.40	754.59	102.81	120.73
32.58	1,016.38	990.38	134.94	158.46
43.43	1,161.58	1,131.86	154.22	181.10
48.86	1,197.87	1,167.24	159.04	186.76
54.29	1,209.98	1,179.03	160.64	188.64
Resisting Section	**Precast Section**		**Composite Section**	

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Dead Load Shears

Distance from the bearing centerline, x (ft.)	Shears at distance x from bearing centerline			
	Loading			
	Girder self weight, V_g (kips)	Slab weight, V_S (kips)	Barrier weight, V_{barr} (kips)	Wearing surface weight, V_{ws} (kips)
0.00	44.57	43.43	5.92	6.95
2.88	42.21	41.13	5.60	6.58
10.86	35.66	34.75	4.73	5.56
21.72	26.74	26.06	3.55	4.17
32.58	17.83	17.37	2.37	2.78
48.86	4.46	4.34	0.59	0.69
54.29	0.00	0.00	0.00	0.00
Resisting Section	**Precast Section**		**Composite Section**	

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Live Load

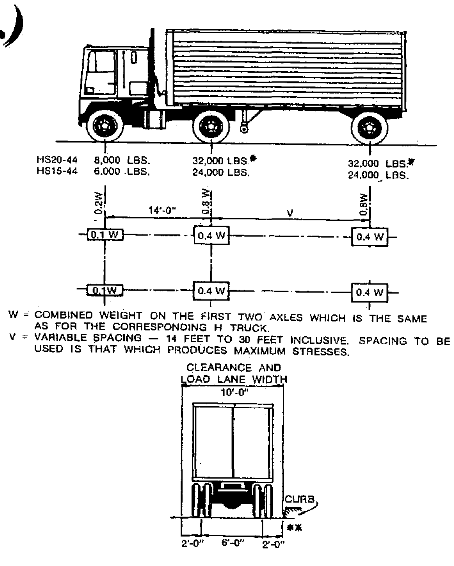
- Design live load for LRFD design shall be taken as HL-93 which consists of the greater of following two combinations [LRFD Art. 3.6]
  - o Combination 1: HS20 design truck + Design lane load
  - o Combination 2: Design tandem + Design lane load

    Design tandem consists of a pair of 25 kip axles spaced 4 ft. apart

    Design lane load consists of a uniform load of 0.64 klf

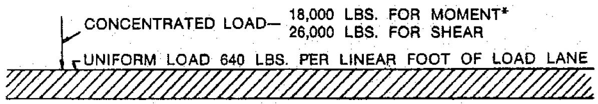
> *The LRFD design live load has changed significantly as compared to the Standard Specifications where the design live load is specified to be taken as the greater of*
> - *HS20 truck load,*
> - *Lane load (consisting of 0.64 klf uniform load and a traversing point load), or*
> - *Tandem load (consisting of a pair of 24 kip axles spaced 4 ft. apart).*

Live Load (cont.)

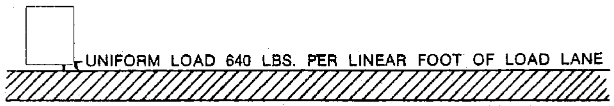


HS20 Truck Configuration

Live Load (cont.)



HS20 Lane Load (for Standard Specifications)



Lane load (for LRFD Specifications)

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Dynamic Load Allowance

- LRFD Art. 3.6.2 specifies a dynamic load allowance of 33% to be applied to truck and tandem loading only. The live load effect including dynamic load effects can be calculated as

$$LL{+}IM = LL(1.33)$$

The impact load factor is specified by Standard Specifications to be taken as

$$I = \frac{50}{L+125} = \frac{50}{108.583+125} = 0.214$$

$$LL{+}I = LL(1.214)$$

This value is 35% smaller than the value of impact load specified by LRFD Specifications

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Live Load Moments

M = Live load moment, k-ft.

x = Distance from the centerline of bearing to the section at which bending moment or shear force is calculated, ft.

L = Design span length = 108.583 ft.

w = Uniform load per linear foot of load lane = 0.64 klf

T = Tandem load = 50 kips

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Live Load Moments (cont.)

- The maximum live load moments due to HS20 truck load are evaluated using the following formulas
- For $x/L = 0 - 0.333$

$$M = \frac{72(x)[(L - x) - 9.33]}{L}$$

- For $x/L = 0.333 - 0.5$

$$M = \frac{72(x)[(L - x) - 4.67]}{L} - 112$$

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Live Load Moments (cont.)

- The live load moments due to lane load are evaluated using the following formula

$$M = 0.5(w)(x)(L\text{-}x)$$

- The live load moments due to tandem load are evaluated using the following formula

$$M = \frac{T(x)[(L - x) - 2]}{L}$$

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Live Load Moments (cont.)

Distance from the bearing centerline x (ft.)	Moments at distance x from bearing centerline				
	Truck Load M_{LT} (k-ft.)	Truck Load + Impact M_{LT+IM} (k-ft.)	Tandem Load M_{LTd} (k-ft.)	Tandem Load + Impact M_{LTd+IM} (k-ft.)	Lane Load M_{LL} (k-ft.)
2.88	183.73	244.36	137.30	182.60	97.25
10.86	636.44	846.47	478.62	636.57	339.56
21.72	1,116.52	1,484.98	848.66	1,128.72	603.66
32.57	1,440.25	1,915.53	1,110.12	1,476.46	792.31
43.43	1,629.82	2,167.66	1,263.00	1,679.78	905.49
48.86	1,671.64	2,223.28	1,298.71	1,727.29	933.79
54.29	1,674.37	2,226.92	1,307.29	1,738.69	943.22

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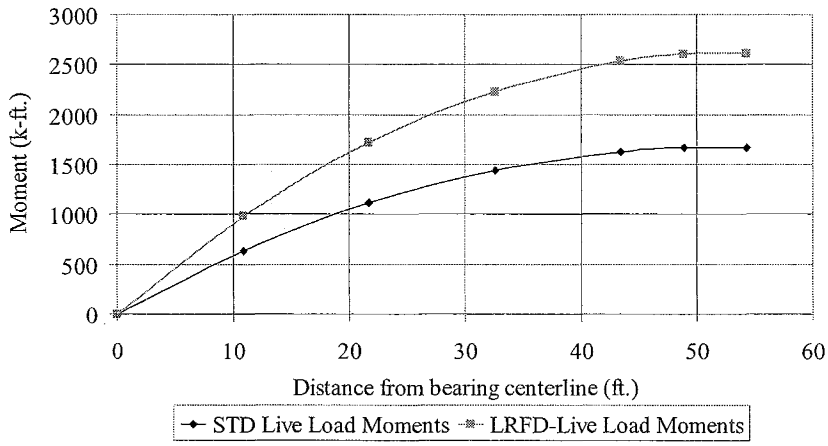

Comparison of Undistributed Live Load Moments

Live Load Shears

- The maximum live load shears due to HS20 truck load are evaluated using the following formulas.

  For $x/L = 0 - 0.5$

$$V = \frac{72[(L - x) - 9.33]}{L}$$

- The live load shears, V, due to lane load are evaluated using the following formula

  For $x \leq 0.5L$ $\quad V = \frac{0.32(L - x)^2}{L}$

Live Load Shears (cont.)

- The live load shears, V, due to tandem load are evaluated using the following formula

$$V = \frac{T[(L - x) - 2]}{L}$$

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Live Load Shears (cont.)

Distance from the bearing centerline x (ft.)	Shear at distance x from bearing centerline				
	Truck Load V_{LT} (kips)	Truck Load + Impact V_{LT+IM} (kips)	Tandem Load V_{LTd} (kips)	Tandem Load + Impact V_{LTd+IM} (kips)	Lane Load V_{LL} (kips)
0.00	65.81	87.53	49.08	65.28	34.75
2.88	63.91	85.00	47.76	63.51	32.93
10.86	58.61	77.96	44.08	58.63	28.14
21.72	51.41	68.38	39.08	51.98	22.24
32.58	44.21	58.80	34.08	45.33	17.03
48.86	33.41	44.44	26.58	35.35	10.51
54.29	29.81	39.65	24.08	32.03	8.69

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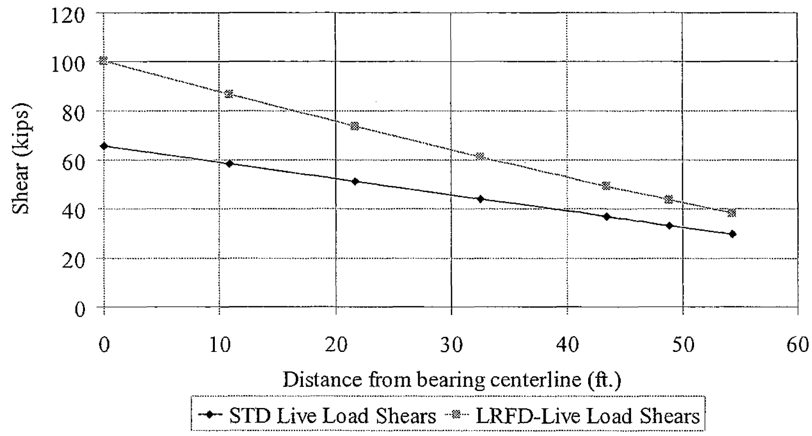

Comparison of Undistributed Live Load Shears

Distribution of Live Load Effects

- The approximate LRFD distribution factors (DFs) may be used if the following conditions are satisfied [LRFD Art. 4.6.2.2]
  - o Width of deck is constant (O.K.)
  - o Number of beams, N_b, is not less than four [$N_b = 6$] (O.K.)
  - o Beams are parallel and have approximately the same stiffness (O.K.)
  - o The roadway part of the overhang, $d_e \leq 3.0$ ft. [d_e = 1'-8"] (O.K.)
  - o Curvature in plan is less than 4° [Curvature = 0°] (O.K.)
  - o Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.2.1-1 [Type k] (O.K.)

Distribution of Live Load Effects (Cont...)

Live Load Moment Distribution Factors for Interior Girders

Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

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Distribution of Live Load Effects (Cont...)

Live Load Shear Distribution Factors for Interior Girders

Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T-and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$

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Parameters for DF Calculations

DFM = Distribution factor for moment
S = Girder spacing = 8 ft.
L = Design span length = 108.583 ft.
t_s = Slab thickness = 8 in.
N_b = Number of girders in the cross section = 6
n = Modular ratio between slab and girder concrete = 1
I = Moment of inertia of the girder section = 260,403 in.^{3}
A = Area of the girder cross section = 788.4 in.^{2}
e_g = Distance between the centroid of the girder and the slab, in.
= $(t_s/2) + y_t$ = (8 in./2 + 29.25 in.) = 33.25 in.
K_g = Longitudinal stiffness parameter, in.^{4} = $n\,(I + Ae_g^{\,2})$
= $1[260{,}403 + (788.4)(33.25)^2] = 1{,}132{,}028.5$ in.^{4}

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Additional Requirements for Type k Girders

- The following requirements must also be satisfied to use the approximate LRFD DF formulas for Type k girders

• 3.5 ft. $\leq S \leq$ 16.0 ft.	S = 8 ft.	(O.K.)
• 4.5 in. $\leq t_s \leq$ 12.0 in.	t_s = 8 in.	(O.K.)
• 20 ft. $\leq L \leq$240 ft.	L = 108.58 ft.	(O.K.)
• $N_b \geq 4$	$N_b = 6$	(O.K.)
• $10{,}000 \leq K_g \leq 7{,}000{,}000$	K_g = 1,132,028.5 in.^{4}	(O.K.)

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Moment Distribution Factors

- One design lane loaded
  [LRFD Table 4.6.2.2.2b-1 girder cross-section type k]

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

$$DFM = 0.06 + \left(\frac{8}{14}\right)^{0.4}\left(\frac{8}{108.583}\right)^{0.3}\left(\frac{1132028.5}{12.0(108.583)(8)^3}\right)^{0.1}$$

= 0.445 lanes/girder

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Moment Distribution Factors

- Two or more design lanes loaded
  [LRFD Table 4.6.2.2.2b-1 girder cross-section type k]

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

$$DFM = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{108.583}\right)^{0.2}\left(\frac{1132028.5}{12.0(108.583)(8)^3}\right)^{0.1}$$

= 0.639 lanes/girder

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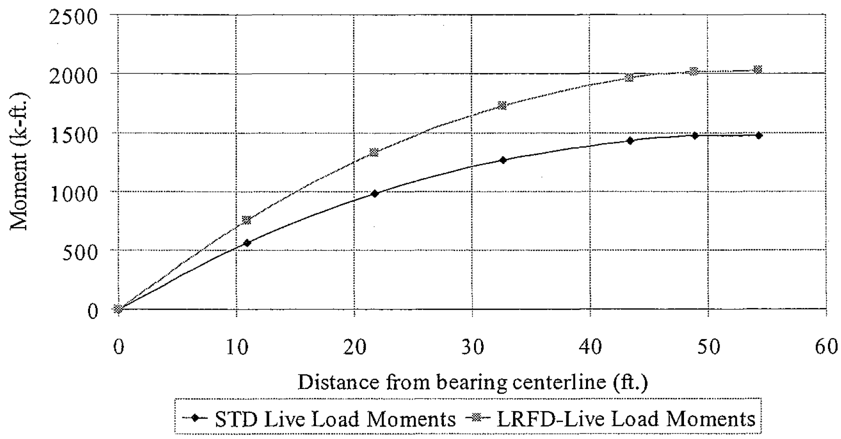
Moment Distribution Factors

- Therefore, the distribution factor for moment, $DFM = 0.639$ shall be used.

- *The Standard Specifications recommend using a DF of S/11, where S is the girder spacing in feet. This gives a DF of 0.727 (13.8% greater).*

Distributed Live Load Moments

Distance from the bearing centerline x (ft.)	Moments at distance x from bearing centerline		
	Truck Load + Impact M_{LT+IM} (k-ft.)	Tandem Load + Impact M_{LTd+IM} (k-ft.)	Lane Load M_{LL} (k-ft.)
2.88	156.15	116.68	62.14
10.86	540.89	406.77	216.98
21.72	948.90	721.25	385.74
32.57	1224.02	943.46	506.28
43.43	1385.13	1073.38	578.61
48.86	1420.68	1103.74	596.69
54.29	1423.00	1111.02	602.72



Comparison of Distributed Live Load Moments Including Impact

Shear Distribution Factors

- One design lane loaded
  [Table 4.6.2.2.3a-1 girder cross-section type k]

$$DFV = 0.36 + \left(\frac{S}{25.0}\right) = 0.36 + \left(\frac{8}{25.0}\right) = 0.68$$

- Two or more design lanes loaded

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2 = 0.2 + \left(\frac{8}{12}\right) - \left(\frac{8}{35}\right)^2 = 0.814$$

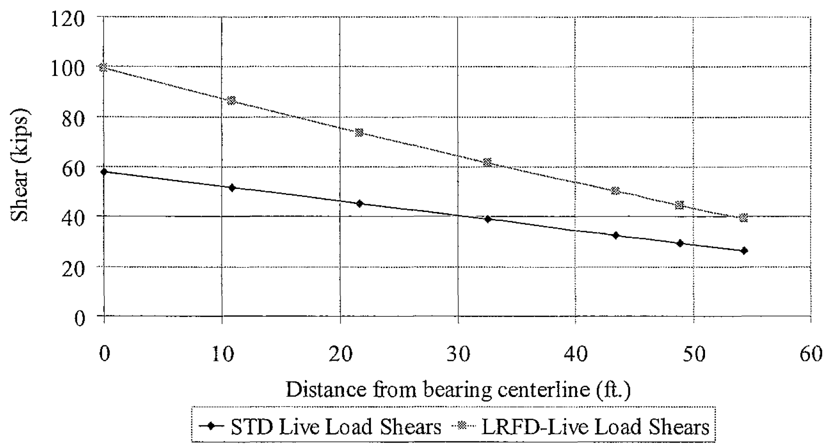
Shear Distribution Factors

- Therefore, the distribution factor for shear, $DFV = 0.814$ shall be used.

- *The Standard Specifications recommend using a DF of S/11, where S is the girder spacing. This gives a DF of 0.727 (10.7% smaller).*

Distributed Live Load Shears

Distance from the bearing centerline x (ft.)	Shear at distance x from bearing centerline		
	Truck Load + Impact V_{LT+IM} (kips)	Tandem Load + Impact V_{LTd+IM} (kips)	Lane Load V_{LL} (kips)
0.00	71.25	53.13	28.28
2.88	69.19	51.70	26.81
10.86	63.46	47.72	22.91
21.72	55.66	42.31	18.10
32.58	47.87	36.89	13.86
48.86	36.17	28.78	8.56
54.29	32.28	26.07	7.07



Comparison of Distributed Live Load Shears Including Impact

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Load Combinations

- The total factored load effect is taken as:

$$Q = \sum \eta i \ \gamma i \ Qi \qquad \text{[LRFD Eq. 3.4.1-1]}$$

Q = Factored force effects.

γ_i = Load factor, a statistically based multiplier applied to force effects specified by LRFD Table 3.4.1-1.

Q_i = Unfactored force effects.

η_i = Load modifier, a factor relating to ductility, redundancy and operational importance.

= $\eta_D \, \eta_R \, \eta_I \geq 0.95$, for loads for which a maximum value of γi is appropriate [LRFD Eq. 1.3.2.1-2]

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Load Combinations (Cont.)

η_D = A factor relating to ductility
= 1.00 for all limit states except strength limit state
= 1.00 for design conventional and complying with the LRFD Specifications is used in this example for strength limit state.

η_R = A factor relating to redundancy
= 1.00 for all limit states except strength limit state.
= 1.00 for designs providing conventional level of redundancy to the structure is used in this example for strength limit state.

η_I = A factor relating to operational importance.
= 1.00 for all limit states except strength limit state.
= 1.00 for typical bridges is used in this example for strength limit state.

$\eta_i = \eta_D \, \eta_R \, \eta_I$ = 1.00 for service and strength limit states

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Load Combinations (cont.)

DC Load effects due to dead loads except wearing surface weight
DW Load effects due to wearing surface weight
LL Live load effects
IM Dynamic load effects

- Strength I: To check ultimate strength [LRFD Table 3.4.1-1]

$$Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)$$

Standard Specifications specifies load factor design Group I loading as 1.3(DL) + 2.17(LL + I).

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Load Combinations (cont.)

- Service I: To check *compressive* stress in prestressed concrete components

$$Q = 1.0(DC + DW) + 1.0(LL + IM)$$

This is the same as service load design Group I loading specified by Standard Specifications.

- Service III: To check *tensile* stresses in prestressed concrete members

$$Q = 1.0(DC + DW) + 0.8(LL + IM)$$

Standard Specifications does not specify a different load combination to check tensile stresses.

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Summary of Changes

- Effective flange width calculations have changed
- Specified conditions must be satisfied to uniformly distribute superimposed dead loads
- Critical Section for shear is no longer $h_c/2$
- Live load has changed to HL-93 model, a combination of truck and lane load (or) tandem and lane load whichever governs

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Summary of Changes (cont.)

- Impact load calculations have changed to 33% of live load for all spans
- Distribution factor is no more $S/11$. Approximate formulas provided shall be used as applicable and if not, refined analysis has to be employed.
- Load combinations have changed

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Part II
Service Limit Design

- Service Limit State Design
  - o Initial strand estimate
  - o Prestress losses
  - o Final strand estimate
  - o Final estimate of concrete strengths
- Summary of changes

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Service Limit State Design

- Design Steps (based on TxDOT methodology)
  - Calculate the tensile stress in the bottom fiber of the girder at midspan section due to service loads using Service III load combination.
  - Calculate allowable tensile stress limit at service limit state.
  - Determine the required precompressive stress in the bottom fiber of the girder. This is the difference between the bottom fiber stress due to applied loads and allowable stress limit.
  - Establish a preliminary estimate of the required number of strands, based on assumed initial prestress loss and prestressing strand eccentricity values.
  - Calculate actual strand eccentricity for the determined number of strands.

Service Limit State Design (Cont.)

- Check if the bottom fiber stress due to prestressing is greater than the required precompressive stress, if not update the number of strands.
- Calculate initial and final prestress losses.
- Calculate the final stress due to prestressing at the bottom fiber of the girder at midspan. Check if this is greater than the required precompressive stress, if not update the number of strands . The number of strands obtained in this step will not be updated any further and will be the final required number of strands.
- Calculate the initial stress at bottom fiber of the girder at the hold down points and estimate the required concrete strength at release using the allowable compression limit at transfer.

Service Limit State Design (Cont.)

- Refine the prestress losses based on the determined required concrete strength at transfer (Prestress loss due to elastic shortening depends on concrete strength at transfer).
- Evaluate the initial stresses at the top and bottom fibers of the girder at the hold down point and girder ends and update the required concrete strength at transfer using allowable stress limits.
- Evaluate the final stresses at the top and bottom fiber of the girders at the midspan and update the required concrete strength at service using allowable stress limits.
- Repeat the above three steps until the required concrete strength at transfer and at service are sufficiently converged.

Service Limit State Design (Cont.)

- Once the required concrete strengths are finalized
  - Check the initial stresses at the top and bottom fiber of the girder at girder end, transfer length, hold down point, and midspan
  - Check the final stresses at the top and bottom fiber of the girder at midspan

Initial Strand Estimate

- Tensile stress at bottom fiber of the girder at midspan due to applied dead and live loads using load combination Service III is given as

$$f_b = \frac{M_{DCN}}{S_b} + \frac{M_{DCC} + M_{DW} + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

M_{DCN} = Moment due to non-composite dead loads, k-ft.
$= M_g + M_S$
M_g = Moment due to girder self-weight = 1,209.98 k-ft.
M_S = Moment due to slab weight = 1,179.03 k-ft.
M_{DCN} = 1,209.98 + 1,179.03 = 2,389.01 k-ft.
M_{DCC} = Moment due to composite dead loads except wearing surface load $= M_{barr}$
M_{barr} = Moment due to barrier weight = 160.64 k-ft.

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Initial Strand Estimate (Cont.)

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.
M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.
M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

- Substituting the moments and section modulus values in the equation

$$f_b = 4.125 \text{ ksi}$$

(*This value is slightly greater than the tensile stress at the bottom fiber of the girder,* 4.024 ksi, *obtained in the Standard design*)

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Initial Strand Estimate (Cont.)

- Allowable tensile stress in fully prestressed concrete members is specified in LRFD Table 5.9.4.2.2-1.
- For members with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions, the allowable tensile stress at service limit state after losses is given as

$$F_b = 0.19\sqrt{f'_c}$$

f'_c = Compressive strength of girder concrete at service = 5 ksi

$$F_b = 0.19\sqrt{5.0} = 0.4248 \text{ ksi}$$

(*This value is slightly greater than the allowable tensile stress,* 0.4242 ksi, *obtained in the Standard design*)

Initial Strand Estimate (Cont.)

- Required precompressive stress

  $f_{pb\text{-}reqd.}$ = Bottom tensile stress – Allowable tensile stress at service

  $= f_b - F_b = 4.125 - 0.4248 = 3.700$ ksi
- The eccentricity of prestressing strands is assumed to be equal to the distance from the centroid of the girder to the bottom fiber

  $e_c = y_b = 24.75$ in. (PSTRS 14 methodology, TxDOT 2004)
- Stress at bottom fiber of the girder due to prestress after losses:

$$f_{b-req'd} = \frac{P_{pe}}{A} + \frac{P_{pe}\, e_c}{S_b}$$

P_{pe} = Prestressing force after all losses, kips

A = Area of girder cross-section = 788.4 in.^{2}

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

Initial Strand Estimate (Cont.)

- Substituting the corresponding values and solving for P_{pe}

  P_{pe} = 1,021.89 kips

- Assuming final losses equal to 20% of the initial prestress, f_{pi} , the prestressing force per strand after losses

P_e = (area of strand) (f_{pi} – losses) = 0.153[202.5 – 0.2(202.5)] = 24.78 kips

Number of prestressing strands required = $P_{pe}/P_e \approx 42$

Strand eccentricity at midspan after strand arrangement

$$24.75 - \frac{12(2+4+6)+6(8)}{42}$$

Stress at bottom fiber of the girder due to prestressing force

$f_b = 3.316 \text{ ksi} < f_{pb\text{-}reqd.} = 3.700 \text{ ksi}$

Initial Strand Estimate (Cont.)

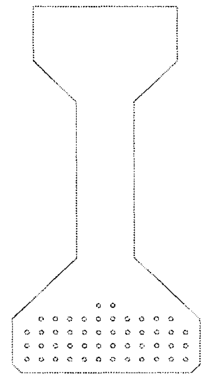
- The strands are incremented by two in each step and the stress at the bottom fiber of the girder due to prestressing is checked until it exceeds the required precompressive stress.

Number of Strands	Prestressing Force, P_{pe} (kips)	Eccentricity at Midspan, e_c (in.)	Stress at Bottom Fiber of the Girder f_b (ksi)
42	1,040.76	20.18	3.316
44	1,090.32	20.02	3.458
46	1,139.88	19.88	3.600
48	1,189.44	19.67	3.723

Initial Strand Estimate (Cont.)

- 48 strands are used as a preliminary estimate for the number of strands. The strand arrangement is shown

Number of Strands	Distance from Bottom Fiber (in.)
2	10
10	8
12	6
12	4
12	2



Initial Strand Arrangement

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Prestress Losses

- The LRFD Specifications specifies the following expressions to be used for the estimation of instantaneous and final losses
- Instantaneous loss of prestress, $\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pR1})$
- Percent instantaneous loss, $\% \, \Delta f_{pi} = \dfrac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$

Δf_{pES} = Prestress loss due to elastic shortening, ksi

Δf_{pR1} = Prestress loss due to steel relaxation before transfer, ksi

f_{pj} = Jacking stress in prestressing strands = 202.5 ksi

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Prestress Losses (Cont.)

The TxDOT methodology is used for the evaluation of instantaneous prestress loss in Standard design, given by the following expression, because Standard Specifications do not provide the expression to evaluate steel relaxation loss at transfer.

$$\Delta f_{pi} = (ES + \frac{1}{2} CR_S)$$

ES = Prestress loss due to elastic shortening, ksi
CR_S = Prestress loss due to steel relaxation at service, ksi

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Prestress Losses (Cont.)

- Time dependent losses: The LRFD Specifications provides two options for the estimation of time dependent losses
  - Lump-sum Estimate
  - **Refined Estimate (used for the detailed design example)**

Time Dependent loss = $\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$

Δf_{pSR} = Prestress loss due to concrete shrinkage, ksi
Δf_{pCR} = Prestress loss due to concrete creep, ksi
Δf_{pR2} = Prestress loss due to steel relaxation after transfer, ksi

- **Total prestress loss:**

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$$

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Prestress Loss due to Elastic Shortening

- The loss in prestress due to elastic shortening in prestressed members is given as

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{[LRFD Eq. 5.9.5.2.3a-1]}$$

The Standard Specifications specify a similar equation for the estimation of elastic shortening loss. However, note that the value for the modulus of elasticity of steel was specified as 28,000 ksi by Standard Specifications. The LRFD Specifications specifies this value as 28,500 ksi.

E_p = Modulus of elasticity of prestressing steel = 28,500 ksi
E_{ci} = Modulus of elasticity of girder concrete at transfer, ksi
= $33{,}000(w_c)^{1.5}\sqrt{f'_{ci}}$ [LRFD Eq. 5.4.2.4-1]

Elastic Shortening (Cont.)

w_c = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.3.4-1 to be applicable) = 0.150 kcf
f'_{ci} = Initial estimate of compressive strength of girder concrete at release = 4 ksi
E_{ci} = $[33{,}000(0.150)^{1.5}\sqrt{4}\,] = 3{,}834.25$ ksi
f_{cgp} = Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self-weight of the member at sections of maximum moment, ksi

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

A = Area of girder cross-section = 788.4 in.^{2}
I = Moment of inertia of the non-composite section = 260,403 in.^{4}
e_c = Eccentricity of the prestressing strands at the midspan = 19.67 in.
M_g = Moment due to girder self-weight at midspan = 1,209.98 k-ft.

Elastic Shortening (Cont.)

- The effective prestress after initial losses is unknown at this point. Hence, using the TxDOT methodology initial loss is assumed to be 8% of initial prestress, f_{pi}.

P_i = Pretension force after allowing for the 8% initial loss, kips
= (number of strands)(area of each strand)[$0.92(f_{pi})$]
= 48 (0.153)(0.92)(202.5) = 1,368.19 kips

$$f_{cgp} = \frac{1{,}368.19}{788.4} + \frac{1{,}368.19(19.67)^2}{260{,}403} - \frac{1{,}209.98(12\text{ in./ft.})(19.67)}{260{,}403}$$
$$= 2.671 \text{ ksi}$$

- **Prestress loss due to elastic shortening is**

$$\Delta f_{pES} = \left[\frac{\mathbf{28{,}500}}{\mathbf{3{,}834.25}}\right]\mathbf{(2.671) = 19.854\ ksi}$$

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Prestress Loss due to Concrete Shrinkage

- The loss is prestress due to concrete shrinkage for pretensioned concrete members is given as:

$\Delta f_{pSR} = 17 - 0.15\, H$ [LRFD Eq. 5.9.5.4.2-1]

H = Average annual ambient relative humidity = 60%

$\Delta f_{pSR} = [17 - 0.15(60)] = 8.0$ ksi

A similar expression is specified by the Standard Specifications for the estimation of prestress loss due to concrete shrinkage.

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Prestress Loss due to Concrete Creep

- **The loss in prestress due to creep of concrete is given as:**

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0 \qquad \textbf{[LRFD Eq. 5.9.5.4.3-1]}$$

A similar expression is specified by the Standard Specifications for the estimation of prestress loss due to concrete creep

Δf_{cdp} = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as f_{cgp}.

$$= \frac{M_S\, e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

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Creep Loss (Cont.)

M_S = Moment due to slab weight at midspan section = 1,179.03 k-ft.

M_{SDL} = Moment due to superimposed dead load

= $M_{barr} + M_{DW}$

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

M_{SDL} = 160.64 + 188.64 = 349.28 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from the centroid of the prestressing strands at midspan to the bottom fiber of the girder

= 24.75 – 19.67 = 5.08 in.

I = Moment of inertia of the non-composite section = 260,403 in.^{4}

I_c = Moment of inertia of composite section = 694,599.5 in.^{4}

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Creep Loss (Cont.)

$$\Delta f_{cdp} = \frac{1{,}179.03(12\text{ in./ft.})(19.67)}{260{,}403} + \frac{(349.28)(12\text{ in./ft.})(41.157 - 5.08)}{694{,}599.5}$$

$$= 1.069 + 0.218 = 1.287 \text{ ksi}$$

- **Prestress loss due to creep of concrete is**

  $$\Delta f_{pCR} = 12(2.671) - 7(1.287) = 23.05 \text{ ksi}$$

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Prestress Loss due to Steel Relaxation

- For pretensioned members with low-relaxation prestressing steel, initially stressed in excess of $0.5f_{pu}$, the prestress loss due to steel relaxation at transfer is given as:

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right]f_{pj} \qquad \text{[LRFD Eq. 5.9.5.4.4b-2]}$$

The Standard Specifications does not specify the expression for the estimation of prestress loss due to steel relaxation at transfer. However, TxDOT uses half the final relaxation loss as the initial relaxation loss.

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Steel Relaxation Loss (Cont.)

Δf_{pR1} = Prestress loss due to relaxation of steel at transfer, ksi

f_{pu} = Ultimate stress in prestressing steel = 270 ksi

f_{pj} = Initial stress in tendon at the end of stressing

= $0.75 f_{pu} = 0.75(270) = 202.5$ ksi > $0.5 f_{pu} = 135$ ksi

t = Time estimated in days from stressing to transfer taken as 1 day [default value for PSTRS14 design program (TxDOT 2004)]

f_{py} = Yield strength of prestressing steel = 243 ksi

Prestress loss due to initial steel relaxation is

$$\Delta f_{pR1} = \frac{\log(24.0)(1)}{40}\left[\frac{202.5}{243} - 0.55\right]202.5 = 1.98 \text{ ksi}$$

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Steel Relaxation Loss (Cont.)

- For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

Δf_{pR2} = 30% of [20.0 – 0.4 Δf_{pES} – 0.2($\Delta f_{pSR} + \Delta f_{pCR}$)

[LRFD Art. 5.9.5.4.4c-1]

The Standard Specifications specify a similar equation for the estimation of prestress loss due to steel relaxation after transfer.

$$\Delta f_{pR2} = 0.3[20.0 - 0.4(19.854) - 0.2(8.0 + 23.05)] = 1.754 \text{ ksi}$$

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Instantaneous Loss

- The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pRI}$$
$$= 19.854 + 1.980 = 21.834 \text{ ksi}$$

- The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pj}} = \frac{100(19.854 + 1.980)}{202.5}$$
$$= 10.78\% > 8\% \text{ (assumed value of initial prestress loss)}$$

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Prestress Losses

- The prestress losses are recalculated using the initial prestress loss value obtained in the previous trial. This procedure is repeated until the difference in the initial prestress loss values obtained by two consecutive trials is less than 0.10%. The following Table summarizes the results from different trials.

Trial	Elastic Shortening (ksi)	Concrete Shrinkage (ksi)	Concrete Creep (ksi)	Initial Steel Relaxation (ksi)	Final Steel Relaxation (ksi)	Initial Prestress Loss (ksi)	Initial Loss (%)
1	19.85	8.0	23.05	1.98	1.75	21.83	10.78
2	19.01	8.0	21.68	1.98	1.94	20.99	10.37
3	19.13	8.0	21.88	1.98	1.91	21.11	10.42

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Final Strand Estimate

- **Total final loss in prestress**

  $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pES} = Prestress loss due to elastic shortening = 19.13 ksi
Δf_{pSR} = Prestress loss due to concrete shrinkage = 8.0 ksi
Δf_{pCR} = Prestress loss due to concrete creep = 21.88 ksi
Δf_{pR1} = Prestress loss due to steel relaxation at transfer = 1.98 ksi
Δf_{pR2} = Prestress loss due to steel relaxation after transfer = 1.91 ksi

$\Delta f_{pT} = 19.13 + 8.0 + 21.88 + 1.98 + 1.91 = \underline{52.90 \text{ ksi}}$

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Final Strand Estimate (Cont.)

- Effective final prestress

  $f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 52.901 = \underline{149.60 \text{ ksi}}$

- Check for prestressing stress limit at service limit state: $f_{pe} \leq 0.8 f_{py}$

  f_{py} = Yield strength of prestressing steel = 243 ksi

  $f_{pe} = 149.60 \text{ ksi} < 0.8(243) = 194.4 \text{ ksi}$ (O.K.)

- Effective prestressing force after allowing for final prestress loss

  P_{pe} = (number of strands)(area of each strand)(f_{pe})

  = 48(0.153)(149.60) = 1,098.66 kips

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Final Strand Estimate (Cont.)

- Stress at the bottom fiber of the girder due to prestress after losses:

$$f_{bf} = \frac{P_{pe}}{A} + \frac{P_{pe}\, e_c}{S_b}$$

Eccentricity of prestressing strands, e_c = 19.67 in.

Substituting the corresponding values in above equation, the stress at the bottom fiber of the girder is determined as

$$f_{bf} = 3.447 \text{ ksi} < f_{pb\text{-}reqd.} = 3.700 \text{ ksi}$$

- The strands are incremented by two in each step and the stress at the bottom fiber of the girder due to prestressing until it exceeds the required precompressive stress.

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Final Strand Estimate (Cont.)

Number of Strands	Prestressing Force, P_{pe} (kips)	Eccentricity at Midspan, e_c (in.)	Stress at Bottom Fiber of the Girder, f_b (ksi)
48	1,098.66	19.67	3.447
50	1,144.44	19.47	3.570
52	1,190.22	19.29	3.691
54	1,236.00	19.12	3.813

- 54 – ½ in. diameter, 270 ksi low-relaxation strands will be used and this will not be updated any further.

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Final Concrete Strengths

- **Total prestress loss at transfer**

$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pR1})$

$= 19.13 + 1.98 = 21.11$ ksi

- **Effective initial prestress**

$f_{pi} = 202.5 - 21.11 = 181.39$ ksi

P_i = Effective pretension after allowing for the initial prestress loss
= (number of strands)(area of each strand)(f_{pi})
= 54(0.153)(181.39) = 1,498.64 kips

Initial Stress at the Hold Down Point

- The concrete stress at release is updated based on the initial stress at the bottom fiber of the girder at the hold down point due to effective initial prestress and self-weight of the girder.

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

Note that PSTRS 14 program uses the design span length for the evaluation of initial stresses. However, this design example uses the overall girder length for initial stress calculations assuming that the girder rests on the ground at transfer for application of self-weight.

Initial Stress at the Hold Down Point (Cont.)

M_g = Moment due to girder self-weight at the hold down point based on overall girder length of 109'-8" = $0.5wx(L - x)$

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Distance of hold down point from the end of the girder
= HD + (distance from centerline of bearing to the girder end)

HD = Hold down point distance from centerline of the bearing
= 48.862 ft.

x = 48.862 + 0.542 = 49.404 ft. (refer girder end details)

M_g = 0.5(0.821)(49.404)(109.67 - 49.404) = 1,222.22 k-ft.

Initial Stress at the Hold Down Point (Cont.)

- Initial concrete stress at bottom fiber of the girder at the hold down point

$$f_{bi} = \frac{1,498.64}{788.4} + \frac{1,498.64(19.12)}{10,521.33} - \frac{1,222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.901 + 2.723 - 1.394 = 3.230 \text{ ksi}$$

- Compression stress limit for pretensioned members at transfer is 0.6 f'_{ci}
[LRFD Art. 5.9.4.1.1]

Therefore, $f'_{ci\text{-}reqd.} = \frac{3,230}{0.6} = 5,383.33$ psi

Refined Losses

- The prestress losses are refined based on the updated number of strands and concrete strength at release. The same approach as discussed in the "Prestress Losses" slides is used. The initial estimate for the initial prestress loss is taken as 10.42%, obtained in the previous trial.

Trial	Elastic Shortening (ksi)	Concrete Shrinkage (ksi)	Concrete Creep (ksi)	Initial Steel Relaxation (ksi)	Final Steel Relaxation (ksi)	Initial Prestress Loss (ksi)	Initial Loss (%)
1	18.83	8.0	26.50	1.98	1.67	20.81	10.28
2	18.87	8.0	26.57	1.98	1.66	20.85	10.30

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Total Initial Prestress Loss

- **Total prestress loss at transfer**

$$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pR1})$$
$$= 18.87 + 1.98 = 20.85 \text{ ksi}$$

- **Effective initial prestress**

$f_{pi} = 202.5 - 20.85 = 181.65$ ksi

P_i = Effective pretension after allowing for the initial prestress loss
= (number of strands)(area of each strand)(f_{pi})
= 54(0.153)(181.65) = 1,500.79 kips

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Total Final Losses

- **Total final loss in prestress**

  $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pES} = Prestress loss due to elastic shortening = 18.87 ksi
Δf_{pSR} = Prestress loss due to concrete shrinkage = 8.0 ksi
Δf_{pCR} = Prestress loss due to concrete creep = 26.57 ksi
Δf_{pR1} = Prestress loss due to steel relaxation at transfer = 1.98 ksi
Δf_{pR2} = Prestress loss due to steel relaxation after transfer = 1.66 ksi

$\Delta f_{pT} = 18.87 + 8.0 + 26.57 + 1.98 + 1.66 = \underline{57.08 \text{ ksi}}$

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Effective Final Prestress

- Effective final prestress

  $f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 57.08 = \underline{145.42 \text{ ksi}}$

- Check for prestressing stress limit at service limit state: $f_{pe} \leq 0.8 f_{py}$

  f_{py} = Yield strength of prestressing steel = 243 ksi

  $f_{pe} = 145.42 \text{ ksi} < 0.8(243) = 194.4 \text{ ksi}$ (O.K.)

- Effective prestressing force after allowing for final prestress loss

  P_{pe} = (number of strands)(area of each strand)(f_{pe})

  = 54(0.153)(145.42) = 1,201.46 kips

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Final Stresses at Midspan

- The required concrete strength at service is updated based on the final stresses at the top and the bottom fibers of the girder at midspan section.
- The concrete stress at the top fiber of the girder at the midspan section is investigated for the following three cases using Service I limit state
  - Case I: Effective final prestress + Permanent loads
  - Case II: Live load + ½ (effective final prestress + permanent loads)
  - Case III: Effective final prestress + Permanent loads + Live load
- The concrete stress at the bottom fiber of the girder at the midspan section is investigated using Service III limit state (The live loads are multiplied by 0.8)

Final Stresses at Midspan (Cont.)

Load	Top Fiber (ksi)	Bottom Fiber (ksi)	Allowable Stress Limit	Required Concrete Strength (psi)
Effective Prestress + Permanent Loads	2.241	-	$0.45\ f'_c$	**4,980**
Live Load + ½ (Effective Prestress + Permanent Loads)	1.570	-	$0.40\ f'_c$	3,925
Effective Prestress + Permanent Loads + Live Load	2.690	-	$0.60\ f'_c$	4,483
Effective Prestress + Permanent Loads + 0.8(Live Load)	-	– 0.418	$0.19\sqrt{f'_c}$	4,840

Initial Stresses

- The initial stresses at the top and bottom fiber of the girders is calculated at the hold down points and girder ends.
- The 10 web strands are harped at the girder end to minimize the initial stresses at the girder end. (See LRFD detailed example pg. A.2-51 for detailed discussion)
- Eccentricity at the girder end is calculated as follows

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54}$$

$= 11.34$ in.

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Initial Stresses (Cont.)

Location		Stress	Allowable Stress Limit	Required Concrete Strength (psi)
Hold Down Points	Top Fiber	0.328	$0.60\,f'_c$	547
	Bottom Fiber	3.237	$0.60\,f'_c$	5,395
Girder End	Top Fiber	– 0.008	$0.24\sqrt{f'_c}$	1
	Bottom Fiber	3.522	$0.60\,f'_c$	**5,870**

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Refined Losses

- The concrete strength at release is updated and the prestress losses are calculated based on the updated concrete strength at release.

Trial	Elastic Shortening (ksi)	Concrete Shrinkage (ksi)	Concrete Creep (ksi)	Initial Steel Relaxation (ksi)	Final Steel Relaxation (ksi)	Initial Prestress Loss (ksi)	Initial Loss (%)
1	18.07	8.0	26.57	1.98	1.76	20.05	9.90
2	18.17	8.0	26.77	1.98	1.73	20.15	9.95

Total Initial Prestress Loss

- **Total prestress loss at transfer**

$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pR1})$

$= 18.17 + 1.98 = 20.15$ ksi

- **Effective initial prestress**

$f_{pi} = 202.5 - 20.15 = 182.35$ ksi

P_i = Effective pretension after allowing for the initial prestress loss
= (number of strands)(area of each strand)(f_{pi})
= 54(0.153)(182.35) = 1,506.58 kips

Total Final Losses

- **Total final loss in prestress**

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$$

Δf_{pES} = Prestress loss due to elastic shortening = 18.17 ksi
Δf_{pSR} = Prestress loss due to concrete shrinkage = 8.0 ksi
Δf_{pCR} = Prestress loss due to concrete creep = 26.77 ksi
Δf_{pR1} = Prestress loss due to steel relaxation at transfer = 1.98 ksi
Δf_{pR2} = Prestress loss due to steel relaxation after transfer = 1.73 ksi

Δf_{pT} = 18.17 + 8.0 + 26.77 + 1.98 + 1.73 = 56.70 ksi

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Effective Final Prestress

- Effective final prestress

  $f_{pe} = f_{pi} - \Delta f_{pT}$ = 202.5 – 56.70 = 145.80 ksi

- Check for prestressing stress limit at service limit state: $f_{pe} \leq 0.8 f_{py}$

  f_{py} = Yield strength of prestressing steel = 243 ksi

  f_{pe} = 145.80 ksi < 0.8(243) = 194.4 ksi (O.K.)

- Effective prestressing force after allowing for final prestress loss

  P_{pe} = (number of strands)(area of each strand)(f_{pe})

  = 54(0.153)(145.80) = 1,204.60 kips

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Final Stresses at Midspan (Cont.)

Load	Top Fiber (ksi)	Bottom Fiber (ksi)	Allowable Stress Limit	Required Concrete Strength (psi)
Effective Prestress + Permanent Loads	2.238	-	$0.45f'_c$	**4,973**
Live Load + ½ (Effective Prestress + Permanent Loads)	1.568	-	$0.40f'_c$	3,920
Effective Prestress + Permanent Loads + Live Load	2.687	-	$0.60f'_c$	4,478
Effective Prestress + Permanent Loads + 0.8(Live Load)	-	– 0.408	$0.19\sqrt{f'_c}$	4,611

Initial Stresses (Cont.)

Location		Stress	Allowable Stress Limit	Required Concrete Strength (psi)
Hold Down Points	Top Fiber	0.322	$0.60\ f'_c$	537
	Bottom Fiber	3.255	$0.60\ f'_c$	5,425
Girder End	Top Fiber	– 0.008	$0.24\sqrt{f'_c}$	1
	Bottom Fiber	3.535	$0.60\ f'_c$	**5,892**

Final Concrete Strengths

- The concrete strengths have sufficiently converged (22 psi difference). Hence, no more iterations are required.
- Required concrete strength at transfer, f'_{ci} = 5,892 psi
- Required concrete strength at service
  = greater of f'_{ci} and 4,973 psi (obtained from final stresses at midspan)
  f'_c = 5,892 psi
- Required number of ½ in. diameter, 270 ksi low relaxation strands = 54

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Design Summary

- Total initial prestress loss = 9.95%
  (STD = 8.94%, LRFD - increase of 11.3%)
- Total final prestress loss = 28.0 %
  (STD = 25.2%, LRFD - increase of 11%)
- Number of prestressing strands = 54
  (STD = 50 strands, LRFD - increase of 8%)
- Concrete strength at transfer = 5,892 psi
  (STD = 5,455 psi, LRFD - increase of 8%)
- Concrete strength at service = 5,892 psi
  (STD = 5,583 psi, LRFD - increase of 5.5%)

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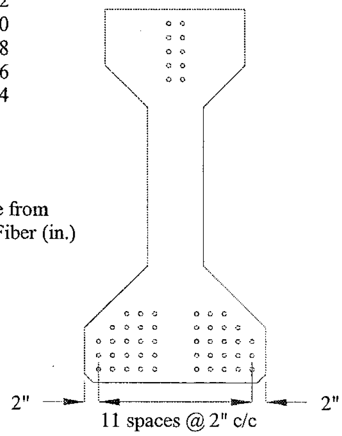
Strand Arrangement

No. of Strands	Distance from Bottom Fiber (in.)
2	52
2	50
2	48
2	46
2	44

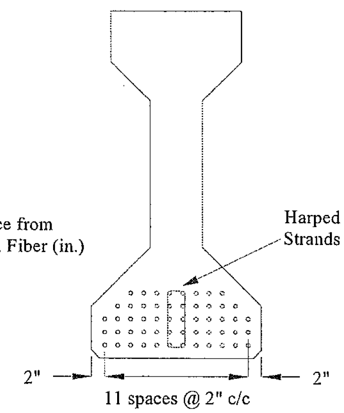
No. of Strands	Distance from Bottom Fiber (in.)
6	10
8	8
10	6
10	4
10	2



Strand Arrangement at Girder End

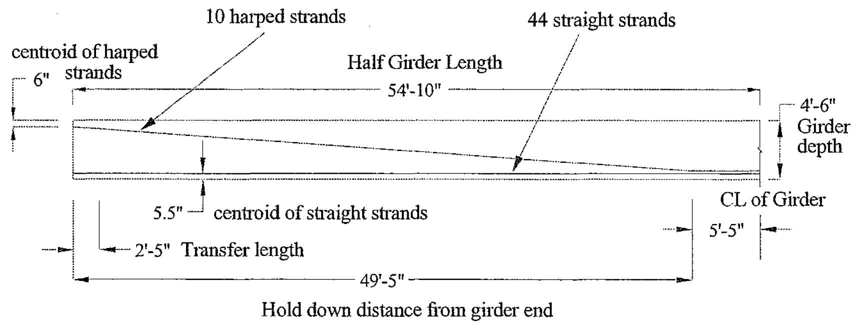
Strand Arrangement (Cont.)

No. of Strands	Distance from Bottom Fiber (in.)
8	10
10	8
12	6
12	4
12	2



Strand Arrangement at Midspan

Strand Arrangement (Cont.)



Longitudinal Strand Profile

Summary of Stresses at Transfer

- **Stresses due to effective initial prestress and self-weight of the girder:**

Location	Top of girder f_t (ksi)	Bottom of girder f_b (ksi)
Girder end	–0.008	+3.535
Transfer length section	+0.074	+3.466
Hold down points	+0.322	+3.255
Midspan	+0.339	+3.241

Summary of Stresses at Service

- **Final stresses at the midspan section for the cases described earlier.**

At Midspan	Top of slab f_t (ksi)	Top of Girder f_t (ksi)	Bottom of girder f_b (ksi)
Case I	+0.126	+2.238	–0.409
Case II	+0.792	+1.568	–
Case III	+0.855	+2.688	–

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Summary of Changes

- The prestress loss due to initial relaxation of steel is included in the LRFD Specifications.
- Allowable stress limit for the compressive stress due to the sum of effective prestress and permanent loads
  - STD: $0.40 f'_c$
  - LRFD: $0.45 f'_c$

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Part III

- Fatigue Limit State Design
- Flexural Strength Design
  - Composite Section properties
  - Check Live Load Moment Distribution Factor
  - Design Moment
  - Moment Resistance
  - Maximum Reinforcement Check
  - Minimum Reinforcement Check
- Summary of Changes

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Fatigue Limit State Design

- The check for the fatigue of the prestressing strands is not required for fully prestressed components designed to have extreme fiber tensile stress due to Service III limit state within the specified limit of $0.19\sqrt{f_c'(ksi)} = 6\sqrt{f_c'(psi)}$

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Composite Section Properties

- The composite section properties are updated using the modular ratio based on chosen concrete strength.
- Modular ratio between slab and girder concrete

$$n = \left(\frac{E_{cs}}{E_{cp}}\right)$$

E_{cs} = Modulus of elasticity of slab concrete = $33{,}000(w_c)^{1.5}\sqrt{f'_{cs}}$
w_c = Unit weight of concrete = 0.150 kcf
f'_{cs} = Compressive strength of slab concrete at service = 4.0 ksi
E_{cs} = $[33{,}000(0.150)^{1.5}\sqrt{4}]$ = 3,834.25 ksi

Composite Section Properties (Cont.)

E_{cp} = Modulus of elasticity of girder concrete at service, ksi
= $33{,}000(w_c)^{1.5}\sqrt{f'_c}$
f'_c = Strength of precast girder concrete at service = 5.892 ksi
E_{cp} = $[33{,}000(0.150)^{1.5}\sqrt{5.892}]$ = 4,653.53 ksi

$$n = \frac{3{,}834.25}{4{,}653.53} = 0.824$$

- Transformed flange width, $b_{tf} = n$*(effective flange width)
  Effective flange width = 96 in.
  b_{tf} = 0.824*(96) = 79.10 in.
- Transformed Flange Area, $A_{tf} = n$*(effective flange width)(t_s)
  t_s = Slab thickness = 8 in.
  A_{tf} = 0.824*(96)(8) = 632.83 in.2

Composite Section Properties (Cont.)

A_c = Total area of composite section = 1,421.23 in.^{2}

I_c = Moment of inertia of composite section = 651,886.0 in^4

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in. = 39.56 in.

y_{tg} = Distance from the centroid of the composite section to extreme top fiber of the precast girder, in. = 54 - 39.56 = 14.44 in.

y_{tc} = Distance from the centroid of the composite section to extreme top fiber of the slab = 62 - 39.56 = 22.44 in.

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.^{3}

= I_c/y_{bc} = 651,886.0/39.56 = 16,478.41 in.^{3}

Composite Section Properties (Cont.)

S_{tg} = Section modulus of the composite section referenced to the top fiber of the precast girder, in.^{3}

= I_c/y_{tg} = 651,886.0/14.44 = 45,144.46 in.^{3}

S_{tc} = Section modulus of composite section referenced to the top fiber of the slab, in.^{3}

= I_c/y_{tc} = 651,886.0/22.44 = 29,050.18 in.^{3}

Live Load Moment Distribution Factor

- Longitudinal stiffness parameter, K_g used in the live load moment distribution factor calculation depends on the modular ratio between girder and slab concrete.
- Live load moment distribution factor calculated using the assumption of modular ratio, n = 1 needs to be checked.

$K_g = n(I + A\, e_g^2)$

n = Modular ratio between girder and slab concrete

$$= \frac{E_c \text{ for girder concrete}}{E_c \text{ for slab concrete}} = \frac{4{,}653.53}{3834.25} = 1.214$$

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Live Load Moment DF (Cont.)

A = Area of non-composite girder cross section = 788.4 in.^{2}

I = Moment of inertia about the centroid of the non-composite precast girder = 260,403 in.^{4}

e_g = Distance between the centers of gravity of the girder and slab = $(t_s/2 + y_t)$ = (8/2 + 29.25) = 33.25 in.

$$K_g = (1.214)[260403 + 788.4\,(33.25)^2] = 1{,}374{,}282.6 \text{ in.}^4$$

$10{,}000 \le K_g \le 7{,}000{,}000$

$10{,}000 \le 1{,}374{,}282.6 \le 7{,}000{,}000$ (O.K.)

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Live Load Moment DF (Cont.)

- One design lane loaded

[LRFD Table 4.6.2.2.2b-1 girder cross-section type k]

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.1}$$

$$DFM = 0.06 + \left(\frac{8}{14}\right)^{0.4}\left(\frac{8}{108.583}\right)^{0.3}\left(\frac{1{,}374{,}282.6}{12.0(108.583)(8)^3}\right)^{0.1}$$

= 0.453 lanes/girder

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Moment Distribution Factors

- Two or more design lanes loaded

[LRFD Table 4.6.2.2.2b-1 girder cross-section type k]

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.1}$$

$$DFM = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{108.583}\right)^{0.2}\left(\frac{1{,}374{,}282.6}{12.0(108.583)(8)^3}\right)^{0.1}$$

= 0.650 lanes/girder

- *DFM* = 0.639 for modular ratio, n = 1, an increase of 1.69%
- Moments need not be updated as the difference is negligible

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Design Moment

- Strength I Load Combination is used for flexural strength design

$$M_u = 1.25(M_{DC}) + 1.5(M_{DW}) + 1.75(M_{LL+IM})$$

M_u = Factored ultimate moment at the midspan, k-ft.

M_{DC} = Moment at the midspan due to dead load of structural components and non-structural attachments, k-ft.
= $M_g + M_S + M_{barr}$

M_g = Moment at the midspan due to girder self-weight
= 1,209.98 k-ft.

M_S = Moment at the midspan due to slab weight = 1,179.03 k-ft.

M_{barr} = Moment at the midspan due to barrier weight = 160.64 k-ft.

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Design Moment (Cont.)

M_{DC} = 1,209.98 + 1,179.03 + 160.64 = 2,549.65 k-ft.

M_{DW} = Moment at the midspan due to wearing surface load
= 188.64 k-ft.

M_{LL+IM} = Moment at the midspan due to vehicular live load including dynamic allowance = $M_{LT} + M_{LL}$

M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.

M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

M_{LL+IM} = 1,423.00 + 602.72 = 2,025.72 k-ft.

- **The factored ultimate bending moment at midspan**
M_u = 1.25(2,549.65) + 1.5(188.64) + 1.75(2,025.72) = 7,015.03 k-ft.

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Moment Resistance

- If $f_{pe} \geq 0.5f_{pu}$, Average stress in the prestressing steel

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

f_{pu} = Specified tensile strength of prestressing steel = 270 ksi

f_{pe} = Effective prestress after final losses = $f_{pj} - \Delta f_{pT}$

f_{pj} = Jacking stress in the prestressing strands = 202.5 ksi

Δf_{pT} = Total final loss in prestress = 56.70 ksi

f_{pe} = 202.5 – 56.70 = 145.80 ksi > $0.5f_{pu}$ = 0.5(270) = 135 ksi

Therefore, the equation for f_{ps} shown above is applicable.

Moment Resistance (Cont.)

k = 0.28 for low-relaxation prestressing strands [LRFD Table C5.7.3.1.1-1]

d_p = Distance from the extreme compression fiber to the centroid of the prestressing tendons = $h_c - y_{bs}$

h_c = Total height of the composite section = 54 + 8 = 62 in.

y_{bs} = Distance from centroid of the prestressing strands at midspan to the bottom fiber of the girder = 5.63 in.

d_p = 62 – 5.63 = 56.37 in.

c = Distance between neutral axis and the compressive face of the section, in.

$$= \frac{A_{ps} f_{pu}}{0.85 f'_c \beta_1 b + k A_{ps}\dfrac{f_{pu}}{d_p}}$$ assuming rectangular section behavior

Moment Resistance (Cont.)

A_{ps} = Area of prestressing steel, in.^{2}
= (number of strands)*(area of each strand)
= (54)(0.153) = 8.262 in.^{2}

f'_c = Compressive strength of deck concrete = 4.0 ksi

β_1 = Stress factor for compression block [LRFD Art. 5.7.2.2]
= 0.85 for ≤ 4.0 ksi

b = Effective width of compression flange = 96 in. (based on non-transformed section)

$$c = \frac{8.262(270)}{0.85(4.0)(0.85)(96) + 0.28(8.262)\left(\frac{270}{56.37}\right)} = 7.73 \text{ in.} < t_s = 8 \text{ in.}$$

The assumption of rectangular section behavior is valid.

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Moment Resistance (Cont.)

Note the change in the definition of rectangular section behavior.

The section can be designed as a rectangular section if

STD $a \leq t_s$

LRFD $c \leq t_s$

a = Depth of equivalent rectangular stress block

c = Depth of neutral axis

t_s *= Depth of compression flange (slab)*

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Moment Resistance (Cont.)

- Average stress in the prestressing steel

$$f_{ps} = 270\left(1 - 0.28\frac{7.73}{56.37}\right) = 259.63 \text{ ksi}$$

The stress in the prestressing steel is 261.57 ksi for Standard design. LRFD – decrease of 0.7%

- Nominal flexural resistance for rectangular section behavior

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

The above equation is a simplified form of LRFD Equation 5.7.3.2.2-1 when mild tension or compression reinforcement is not provided.

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Moment Resistance (Cont.)

a = Depth of the equivalent rectangular compression block, in.
= $\beta_1 c = 0.85(7.73) = 6.57$ in.

- Nominal flexural resistance

$$M_n = (8.262)(259.63)\left(56.37 - \frac{6.57}{2}\right) = 113{,}870.67 \text{ k-in.} = 9{,}489.22 \text{ k-ft.}$$

- Factored flexural resistance:

$$M_r = \phi M_n \qquad \text{[LRFD Eq. 5.7.3.2.1-1]}$$

ϕ = Resistance factor = 1.0 for flexure and tension of prestressed concrete members [LRFD Art. 5.5.4.2.1]

$\phi M_r = (1.0)(9489.22) = 9{,}489.22$ k-ft. $> M_u = 7{,}015.03$ k-ft. (O.K.)

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Maximum Reinforcement Limit

- LRFD Art. 5.7.3.3.1 specifies that the maximum amount of the prestressed and non-prestressed reinforcement should be limited such that

$$\frac{c}{d_e} \leq 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

c = Distance from the extreme compression fiber to the neutral axis = 7.73 in.

d_e = The corresponding effective depth from the extreme fiber to the centroid of the tensile force in the tensile reinforcement

$$= \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} = d_p, \text{ if mild steel tension reinforcement is not used}$$

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Maximum Reinforcement Limit

$$\frac{c}{d_e} = \frac{7.73}{56.37} = 0.137 << 0.42 \quad \text{(O.K.)}$$

The Standard Specifications define a different expression to check the maximum reinforcement limit.

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Minimum Reinforcement Limit

- LRFD Art. 5.7.3.3.2 specifies that at any section of a flexural component, the amount of prestressed and non-prestressed tensile reinforcement should be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:
  - 1.2 times he cracking moment, M_{cr}, determined on the basis of elastic stress distribution and the modulus of rupture of concrete, f_r
  - 1.33 times the factored moment required by the applicable strength load combination.

Minimum Reinforcement Limit

- The above requirements are checked at the midspan section in this design example. Similar calculations can be performed at any section along the girder span to check these requirements.
- The cracking moment, M_{cr}, is given as

$$M_{cr} = S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \le S_c f_r$$

[LRFD Eq. 5.7.3.3.2-1]

Minimum Reinforcement Limit

- f_r =Modulus of rupture, ksi
  = 0.24 for normal weight concrete [LRFD Art. 5.4.2.6]
- f_c' =Compressive strength of girder concrete at service
  =5.892 ksi
- $f_r = 0.24\sqrt{5.892} = 0.582$ ksi
- $f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b}$
- P_{pe} =1,204.60 kips

Minimum Reinforcement Limit

- e_c = 19.12 in.
- A = 788.4 in.^{2}
- S_b = 10,521.33 in.^{3}
- $f_{cpe} = \frac{1,204.60}{788.4} + \frac{1,204.60(19.12)}{10,521.33} = 3.717$ ksi
- M_{dnc}= Total unfactored dead load moment acting on the non-composite section = $M_g + M_S$
- M_g = 1,209.98 k-ft.
- M_S = 1,179.03 k-ft.
- M_{dnc}= 2,389.01 k-ft. = 28,668.12 k-in.

Minimum Reinforcement Limit

- S_{nc} =10,521.33 in.^{3}
- S_c =16,478.41 in.^{3} (based on updated composite section properties)

Minimum Reinforcement Limit

- The cracking moment is:

$$M_{cr} = (16{,}478.41)(0.582 + 3.717) - (28{,}668.12)\left(\frac{16{,}478.41}{10{,}521.33} - 1\right) = 4{,}550.76 \text{ k-ft.}$$

- $S_c f_r$ = (16,478.41)(0.582) = 9,590.43 k-in.
- = 799.20 k-ft. < 4,550.76 k-ft.
- Therefore, use M_{cr} = 799.20 k-ft.

Minimum Reinforcement Limit

- 1.2 M_{cr} = 1.2(799.20) = 959.04 k-ft.
- Factored moment required by Strength I load combination at midspan M_u = 7,015.03 k-ft.
- 1.33 M_u = 1.33(7,015.03 k-ft.) = 9,330 k-ft.
- Since, 1.2 M_{cr} < 1.33 M_u, the 1.2M_{cr} requirement controls.
- M_r = 9,489.22 k-ft >> 1.2 M_{cr} = 959.04 (O.K.)

Summary of Changes

- Maximum reinforcement limit is changed.

Part IV

- Shear Design
  - o Transverse Shear Design
  - o Interface Shear Design
- Summary of Changes

Transverse Shear Design

- LRFD Art. 5.8 specifies shear requirements.
- LRFD Art. 5.8.2.4 specifies – the transverse shear reinforcement is required if:

$$V_u > 0.5\ \phi(V_c + V_p) \qquad \text{[LRFD Art. 5.8.2.4-1]}$$

V_u = Total factored shear force at the section, kips
V_c = Nominal shear resistance of the concrete, kips
V_p = Component of the effective prestressing force in the direction of the applied shear, kips
ϕ = Resistance factor = 0.90 for shear in prestressed concrete members [LRFD Art. 5.5.4.2.1]

Transverse Shear Design (Cont.)

- Critical Section for shear
  - Greater of $0.5d_v\cot\theta$ or d_v

    d_v = Effective shear depth, in.

    = $(d_e - a/2)$, but not less than the greater of $0.9d_e$ or $(0.72h)$ [LRFD Art. 5.8.2.9]

    d_e = Corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement = 56.45 in. [LRFD Art. 5.7.3.3.1]

    a = Depth of compression block = 6.57 in.

Transverse Shear Design (Cont.)

- Effective shear depth

  $d_v = 56.45 - 0.5(6.57) = 53.17$ in. (controls)

  $\geq 0.9d_e = 0.9(56.45) = 50.80$ in. (O.K.)

  $\geq 0.72h = 0.72(62) = 44.64$ in. (O.K.)

  Therefore $d_v = 53.17$ in.

- θ = Angle of inclination of the diagonal compressive stresses.
  - calculated using an iterative process.
  - as an initial estimate take $\theta = 23°$

Transverse Shear Design (Cont.)

- The critical section near the supports is greater of:
  - d_v = 53.17 in. and
  - $0.5d_v\cot\theta = 0.5(53.17)(\cot 23°) = 62.63$ in. from the face of the support *(controls)*

- Add half the bearing width (3.5 in., standard pad size for prestressed girders is 7" × 22") to get the distance of the critical section from the centerline of bearing.
  - $x = 62.63 + 3.5 = 66.13$ in. = 5.51 ft. (0.051L) from the centerline of bearing where L is the design span length.

Transverse Shear Design (Cont.)

- Moments and Shears at Critical section for shear

Load	Girder Self-Weight	Slab	Barrier	Wearing Surface	Live Load + Impact
Moment (k-ft.)	233.54	227.56	31.29	35.84	407.91
Shear (kips)	40.04	39.02	5.36	6.15	92.76
Strength I Load Factors	1.25	1.25	1.25	1.50	1.75

  - Factored shear, V_u = 277.08 kips
  - Factored Moment, M_u = 1383.09 k-ft. $> V_u d_v$ = 1227.69 k-ft. (O.K.)

Transverse Shear Design (Cont.)

- The contribution of the concrete to the nominal shear resistance is given as:

$$V_c = 0.0316\,\beta\,\sqrt{f'_c}\,b_v\,d_v \quad \text{[LRFD Eq. 5.8.3.3-3]}$$

β = A factor indicating the ability of diagonally cracked concrete to transmit tension

b_v = Effective web width taken as the minimum web width within the depth d_v = 8 in.

Determination of θ and β

- Longitudinal strain in the flexural tension reinforcement, assuming minimum transverse reinforcement is provided

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \leq 0.001$$

N_u = Applied factored normal force at the specified section, $0.051L$ = 0 kips

f_{po} = For pretensioned members, this taken as the stress in strands when the concrete is cast around them, which is the jacking stress f_{pj} LRFD = 202.5 ksi [LRFD C5.8.3.4.2]

A_{ps} = Area of straight prestressing strands = 44(0.153) = 6.732 in.^{2}

Determination of θ and β

– Angle of the harped strands to the horizontal

$$\Psi = \tan^{-1}\left(\frac{42.45}{49.4(12\text{in./ft.})}\right) = 0.072 \text{ rad.}$$

V_p = (force per strand)(number of harped strands)(sin Ψ)
= 22.82(10)(sin 0.0072) = 16.42 kips

$$\varepsilon_x = \frac{\frac{1383.09(12\text{ in./ft.})}{53.17} + 0.5(277.08 - 16.42)\cot 23^\circ - 44(0.153)202.5}{2[28000(0.0) + 28500(44)(0.153)]}$$

ε_x = –0.00194 (negative value, LRFD Eq 5.8.3.4.2-3 needs to be used)

Determination of θ and β

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})}$$

A_c = Area of concrete on the flexural tension side below $h_c/2$ = 473 in.^{2}

ε_x = –0.000155

Determination of θ and β

- Shear stress in concrete

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{277.08 - 0.9(16.42)}{0.9(8.0)(53.17)} = 0.685$$

- Interpolate from the table for the obtained values of strain and stress
  - $\theta = 20.47° < 23°$
  - $\beta = 3.20$

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Table 5.8.3.4.2-1 Values of θ and β for Sections with Transverse Reinforcement.

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

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Determination of θ and β

- Refining the critical section based on $\theta = 20.47°$
- Critical section location = $0.057L$
- $V_u = 274.10$ kips
  (as compared to 247.8 kips for Standard Specifications)
- $M_u = 1{,}222.03$ k-ft.
  - $\varepsilon_x = -0.000155$
  - $v_u = 0.677$ ksi
- $\theta = 20.22° \approx 23°$
- $\beta = 3.26$
- $V_c = 106.36$ kips (221.86 kips for Standard)

Shear Reinforcement

- $V_u = 274.10$ kips > 0.5(0.9)(106.36 + 16.42) = 55.25 kips
  - Therefore, transverse shear reinforcement should be provided.

$$V_s = \frac{V_u}{\phi} - V_c - V_p = \left(\frac{274.10}{0.9} - 106.36 - 16.42\right) = 181.77\text{kips}$$

- Area of shear reinforcement within a distance s, for vertical stirrups:
  $Av = (sVs)/\, f_y d_v(\cot\theta + \cot\alpha)\sin\alpha$
  = s(181.77)/(60)(53.17)(cot 20.220 + cot 900) sin 900
  = 0.021(s)
  $A_v = 0.252$ in^2/ft.(for s = 12 in.)

Minimum Shear Reinforcement

- The area of transverse reinforcement should not be less than:

$$0.0316\sqrt{f'_c}\frac{b_v s}{f_y} = 0.0316\sqrt{5.892}\frac{(8)(12)}{60}$$

$$= 0.12 \text{ in.}^2 < A_v = 0.252 \text{ in.}^2$$

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Maximum Nominal Shear Resistance

- The maximum nominal shear resistance V_n shall be such that:

$$V_c + V_s \leq 0.25 f'_c\, b_v d_v$$

$$106.36 + 283.9 = 390.26 \text{ kips}$$

$$\leq 0.25(5.892)(8)(53.17) = 626.55 \text{ kips} \quad \text{O.K.}$$

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Interface Shear Design

- At the strength limit state, the horizontal shear at a section can be calculated as follows

$$V_h = \frac{V_u}{d_v}$$

- The LRFD Specifications do not identify the location of the critical section. For convenience, it will be assumed here to be the same location as the critical section for vertical shear, at point 0.057*L*

Interface Shear Design

- Using load combination Strength I:

  V_u = 1.25(5.29) +1.50(6.06) + 1.75(66.81 + 25.15)

  = 176.63 kips

  dv = 53.17 in

  V_h = 3.30 kip/in.

  Required $V_n = V_h/\varphi = 3.30/0.9 = 3.67$ kip/in

Interface Shear Design

- Calculate the nominal shear resistance at the section

$$V_n = cA_{cv} + \mu[A_{vf}f_y + P_c]$$

A_{cv} = Area of concrete engaged in shear transfer, in.^{2} (taken on a per in. basis as b_v*1 in., where b_v is the width of interface)

= (20)(1) = 20 in.^{2}

A_{vf} = Area of shear reinforcement crossing the shear plane, in.^{2}

f_y = Yield strength of reinforcement = 60 ksi

P_c = Permanent net compressive force normal to the shear plane

= 0 kips

Interface Shear Design

Assuming concrete is placed against hardened concrete clean and free of laitance, but not intentionally roughened.

c = Cohesion factor = 0.075 ksi

μ = Friction factor = 0.6 for normal weight concrete

$$3.67 = (0.075)(20) + 0.6[A_{vf}(60) + 0]$$

Solving for A_{vf} = 0.06 in^2/in or 0.72 in.^{2}/ft.

Minimum $A_{vf} \geq (0.05b_v)/f_y$

A_{vf} = 0.80 in.^{2}/ft. > [0.05(20)/60](12 in./ft) = 0.2 in.^{2}/ft. (O.K.)

Interface Shear Design

$V_n = 3.67$ kip/in. $\leq 0.2 f_c{'} A_{cv} = 16$ kip/in. (O.K.)

$\leq 0.8\, A_{cv} = 16$ kip/in. (O.k.)

Interface shear design is good.

The area of reinforcement required by STD is 0.20 in.^{2}/ft.

Summary of Changes

- The methodology for interface shear and transverse shear changed significantly.
- LRFD provisions are found to be requiring larger area of reinforcement as compared to Standard, significantly larger for interface shear design.

Part V

- Camber and Deflections
- Comparison with Standard Specification Results
- Summary of Changes

Camber and Deflection

- As in Standard , no specific provisions are provided in LRFD for camber and deflection calculations
- Hyperbolic Functions Method used in both the examples
  - An iterative process
- Details not included here as the methodology have not changed
- The camber is found to be 0.425 ft. (STD 0.389 ft.)
- Deflections calculated using elastic analysis
  - Total dead load deflection is computed as 0.138 ft. (STD 0.141 ft.)

Comparison of STD and LRFD Design Examples

Parameter	STD	LRFD	Difference %
Dynamic Load Factor	0.214	0.33	+54.2
Moment DF	0.727	0.639	-12.1
Shear DF	0.727	0.814	+12.0
Initial Prestress Loss	8.94%	9.95%	+11.3
Final Prestress Loss	25.24%	28%	+10.9

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Parameter		STD (psi)	LRFD (psi)	Difference %
Girder Stresses at Transfer				
Girder Ends	Top Fiber	35	-8	**-123.0**
	Bottom Fiber	3,273	3,535	+8.0
Transfer Length	Top Fiber	104	74	-28.8
	Bottom Fiber	3,215	3,466	+7.8
Hold-Down Points	Top Fiber	351	322	-8.3
	Bottom Fiber	3,005	3,255	+8.3
Midspan	Top Fiber	368	339	-7.9
	Bottom Fiber	2,991	3,241	+7.7
Girder Stresses at Service				
Midspan	Top Fiber	2,562	2,688	+4.9
	Bottom Fiber	- 412	- 409	+0.8
Top of Slab Midspan		658	855	+29.9

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Parameter	STD	LRFD	Diff. %
Required Concrete Strength at Transfer	5,455 psi	5,892 psi	+8.0
Required Concrete Strength at Service	5,583 psi	5,892 psi	+5.5
Total Number of Strands	50	54	+8.0
Number of Harped Strands	10	10	0.0
Ultimate Flexural Moment Required	6,769 k-ft.	7015 k-ft.	+3.6
Ultimate Moment Provided	8,936 k-ft	9489 k-ft.	+6.2
Transverse shear Reinf. Area	0.22 in.^{2}	0.252 in.^{2}	+14.5
Interface Shear Reinf. Area	0.2 in.^{2}/ft.	0.72 in.^{2}/ft.	**+260.0**
Maximum Camber	0.389 ft.	0.425ft.	+9.2
Dead Load Deflection	0.141 ft.	0.138 ft.	- 2.1

Summary of Changes

- No change in the methodology for estimating camber and deflection

Impact of LRFD Specifications on the Design of Texas Bridges

Focus is on standard reinforced concrete bents.

TxDOT 0-4751

Research Team

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Project Duration: 9/01/03 – 8/31/05

UTSA

Project Objectives

- Assess the calibration of the current LRFD Specifications with respect to standard reinforced concrete bent caps.
- Identify areas where revisions are needed to provide a more rational approach to design.
- Develop design examples and a hold a seminar to assist in TxDOT's implementation of the LRFD Specifications.

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UTSA

Main Tasks (UTSA/TTI)

1. Review Literature and Current State of Practice
2. Define Prototype Texas Interior Bent Caps
3. Develop Detailed Design Examples
4. Conduct Parametric Study
5. Identify and Address Needs for Revised Design Criteria
6. Complete Final Reports and Recommendations
7. Plan and Conduct Seminar

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UTSA Parametric Study Standard Bent design LRFD and Tx DOT

Parameter	Description / Selected Values
Design Codes	AASHTO Standard Specifications, 17th Ed. (2002) AASHTO LRFD Specifications, 3rd Ed. (2004)
Girder Section	Type IV
Girder Spacing	Type IV 6'-8", 8'-8" and 8'-0
Roadway Widths	24', 30' and 44' Roadways Widths
Spans	40 ft. to 115 ft. span at 5 ft. intervals for Type IV beams
Cap Dimensions	3'3" X 3'6".
f'_c	Class C: 3600 psi
Skew Angle	0, 15, 30 and 45 degrees

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AASHTO LOADS

• PERMANENT LOADS

DD	=	Downdrag
DC	=	Dead load of structural components and non-structural attachments
DW	=	Dead load of wearing surfaces and utilities
EH	=	Horizontal earth pressure load
EL	=	Accumulated locked-in force effects resulting from the construction process, including secondary forces from post-tensioning.

•TRANSIENT LOADS

BR	=	Vehicular braking force
CE	=	Vehicular centrifugal force
CR	=	Creep
CT	=	Vehicular collision force
CV	=	Vessel collision force
EQ	=	Earthquake
FR	=	Friction
IC	=	Ice load
IM	=	Vehicular dynamic load allowance
LL	=	Vehicular live load
LS	=	Live load surcharge
PL	=	Pedestrian live load
SE	=	Settlement
SH	=	Shrinkage
TG	=	Temperature gradient
TU	=	Uniform temperature
WA	=	Water load and stream pressure
WL	=	Wind on live load
WS	=	Wind load on structure

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Loads

	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	EQ	IC	CT	CV
STRENGTH I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH IV	1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH V	γ_p	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE IV	1.00	-	1.00	0.70	-	1.00	1.00/1.20	-	1.0	-	-	-	-
FATIGUE	-	0.75	-	-	-	-	-	-	-	-	-	-	-

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Loads on Standard Bents

	DC *DD* DW *EH* *EV* *ES* *EL*	LL IM *CE* BR *PL* *LS*	WA	WS	WL	*FR*	TU CR SH	*TG*	SE	*EQ*	*IC*	CT	*CV*
STRENGTH I	γ_p	1.75	1.00	-	-	*1.00*	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH II	γ_p	*1.35*	*1.00*	-	-	*1.00*	*0.50/1.20*	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH III	γ_p	-	1.00	1.40	-	*1.00*	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH IV	*1.5*	-	*1.00*	-	-	*1.00*	*0.50/1.20*	-	-	-	-	-	-
STRENGTH V	γ_p	1.35	1.00	0.40	1.00	*1.00*	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT I	γ_p	γ_{EQ}	*1.00*	-	-	*1.00*	-	-	-	*1.00*	-	-	-
EXTREME EVENT II	γ_p	0.50	1.00	-	-	*1.00*	-	-	-	-	*1.00*	1.00	*1.00*
SERVICE I	1.00	1.00	1.00	0.30	1.00	*1.00*	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE II	*1.00*	*1.30*	*1.00*	-	-	*1.00*	*1.00/1.20*	-	-	-	-	-	-
SERVICE III	*1.00*	*0.80*	*1.00*	-	-	*1.00*	*1.00/1.20*	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE IV	*1.00*	-	*1.00*	*0.70*	-	*1.00*	*1.00/1.20*	-	*1.0*	-	-	-	-
FATIGUE	-	*0.75*	-	-	-	-	-	-	-	-	-	-	-

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Loads on Standard Bents (Cont'd)

	DC DW	LL IM BR	WA	WS	WL	TU CR SH	SE	CT
STRENGTH I	γ_p	1.75	1.00	-	-	0.50/1.20	γ_{SE}	-
STRENGTH III	γ_p	-	1.00	1.40	-	0.50/1.20	γ_{SE}	-
STRENGTH V	γ_p	1.35	1.00	0.40	1.00	0.50/1.20	γ_{SE}	-
EXTREME EVENT II	γ_p	0.50	1.00	-	-	-	-	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.00	1.00/1.20	γ_{SE}	-

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Load Factors for Permanent Loads, γ_p

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DW: Wearing Surfaces and Utilities	1.50	0.65

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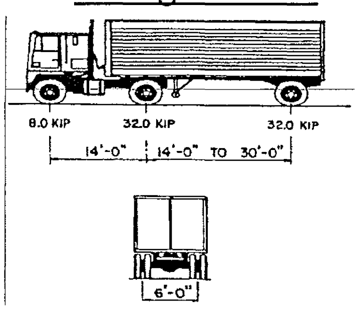
Live load: HL-93

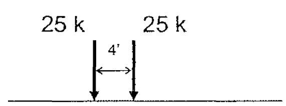
Design Truck



OR

Design Tandem

Two 25.0 kip axles spaced 4.0 ft apart



AND

Design Lane Load

Uniformly distributed load of 0.64 klf

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Application of Design Vehicular Live Load

LRFD 3.6.1.3.1

Service and Strength Limit States:

For negative moment and **reactions at interior piers,** ***consider also the combination of***

- ***90% of the effect of two design trucks with a minimum of 50 FT between the rear axle of the lead truck and the front axle of the second truck. The spacing between 32 KIP axles on each truck shall be 14 FT; and***
- ***90% of the effect of the design lane load***

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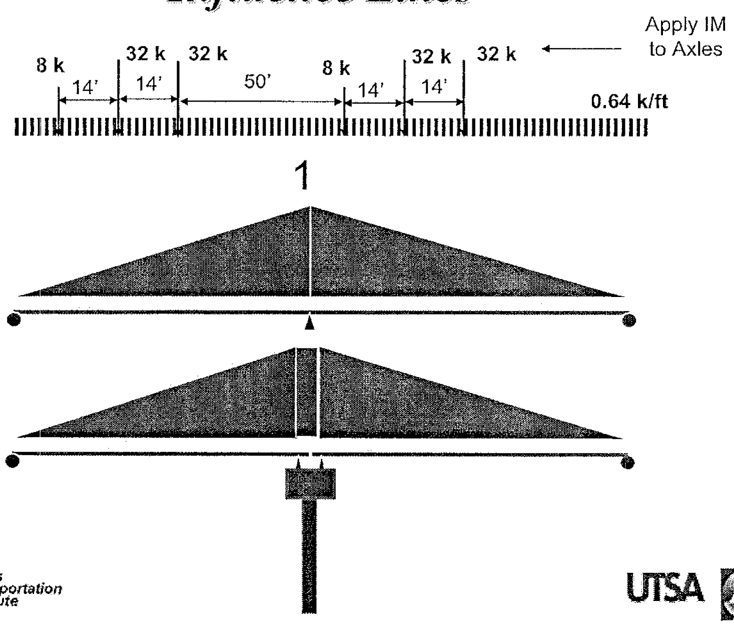
Influence Lines

90% of



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Dynamic Load Allowance, IM

LRFD 3.6.2.1

Component	IM
Deck Joints- All Limit States	75%
All Other Components: • Fatigue and Fracture Limit State • All Other Limit States	33%

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Multiple Presence Factors

Number of Loaded lanes	Multiple Presence Factors (m)
1	1.2
2	1.00
3	0.85
>3	0.65

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Braking Force (BR)
LRFR 3.6.4

The braking force shall be taken as the greater of:

- 25% of the axle weights of the design truck
- 25% of the axle weights of the tandem truck
- 5% of the design truck plus lane load
- 5% of the tandem truck plus lane load

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Braking Force (Cont'd)

- Dynamic Load Allowance (IM) shall not apply (LRFD 3.6.2.1).
- Multiple presence factors shall apply.
- All design lanes carrying traffic headed in the same direction shall be loaded.
- Bridges likely to become one-directional in the future shall have all the design lanes loaded simultaneously.
- Braking force acts horizontally 6 ft. above roadway

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Wind Load on Structure (WS) LRFD 3.8.1.2

- Simplified approach applies to

$$\frac{\textbf{span length}}{\textbf{width}} \le 30 \quad \textbf{and} \quad \frac{\textbf{span length}}{\textbf{depth}} \le 30$$

- Design Equation

$$P_D = P_B \left(\frac{V_{DZ}}{V_B} \right)^2$$

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Wind Load on Structure (Cont'd)

- If $V_{DZ} = V_B = V_{30} = 100$ mph the design equation simplifies to $P_D = P_B$
- Normal to beam or girder spans the total wind load should not be less than 0.30 klf

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Base Wind Pressures, P_D

Skew Angle of Wind (Degrees)	Trusses, Columns and Arches		Girders	
	Lateral Load (ksf)	Longitudinal Load (ksf)	Lateral Load (ksf)	Longitudinal Load (ksf)
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

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Wind Force Applied Directly to Substructure

- Base wind pressure = 0.040 ksf
- For different angles of attack this pressure shall be resolved into components perpendicular to the end and front elevations of the substructure
- Apply simultaneously with wind loads from the superstructure

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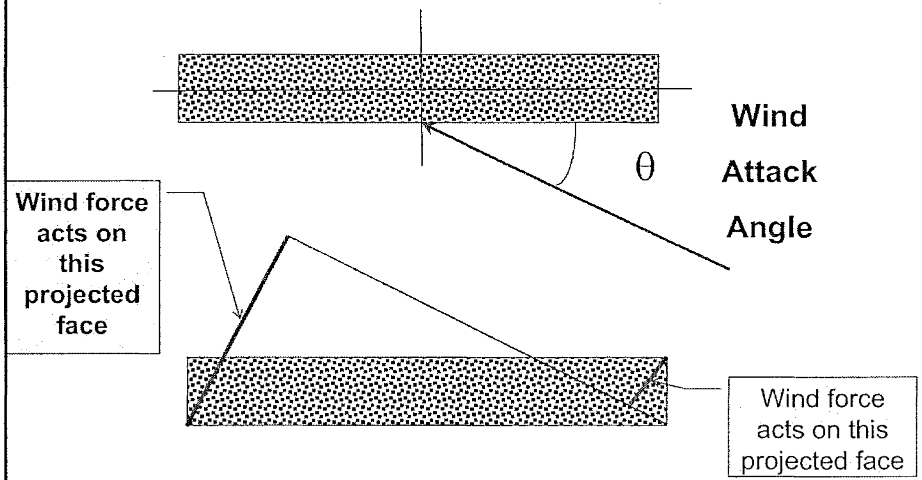
Projected Area for Wind Pressure



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Vertical Wind Load

- Upward Force = 0.020 ksf * Deck Width
- Apply at the windward quarter point of the deck width.
- Apply with other wind loads when the direction of the wind is taken perpendicular to the longitudinal axis of the bridge.
- Do not combine with wind on live load.

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Wind Pressure on Live Load (Vehicles), WL

- Interruptible moving force.
- Applied at 6 ft above roadway.
- Applied only to tributary areas producing the same force effect

Skew Angle (Degrees)	Normal Component (klf)	Parallel Component (klf)
0	0.100	0.000
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

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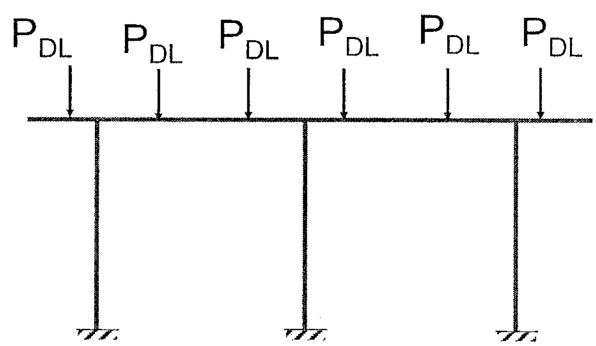
Summary of Loads: Superstructure Dead Loads



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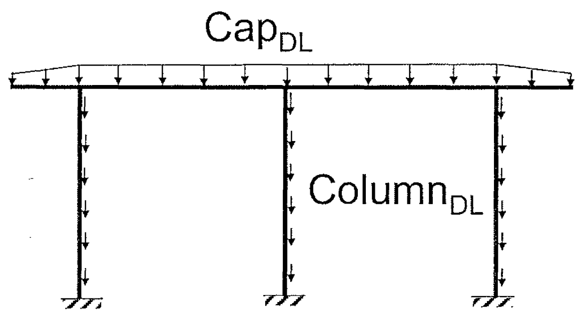
Summary of loads: Substructure dead load



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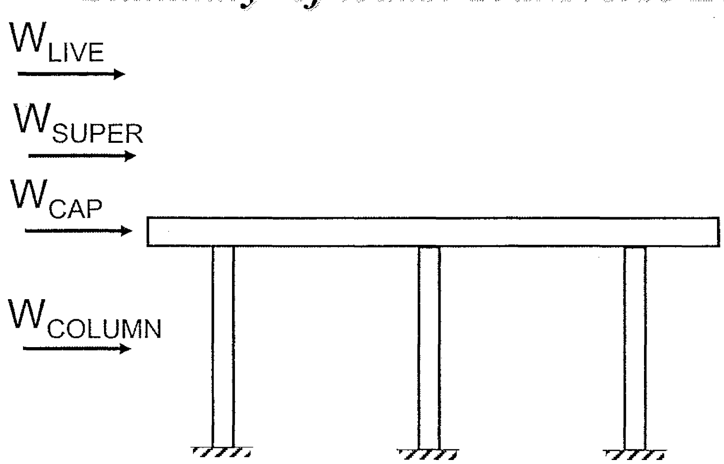
Summary of loads: Transverse Loads



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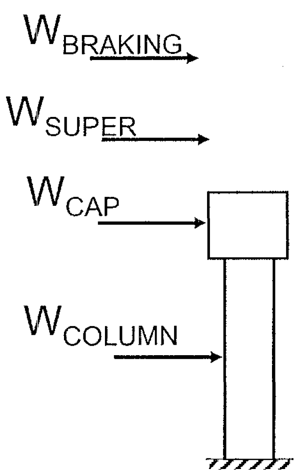
Summary of loads: Longitudinal Loads



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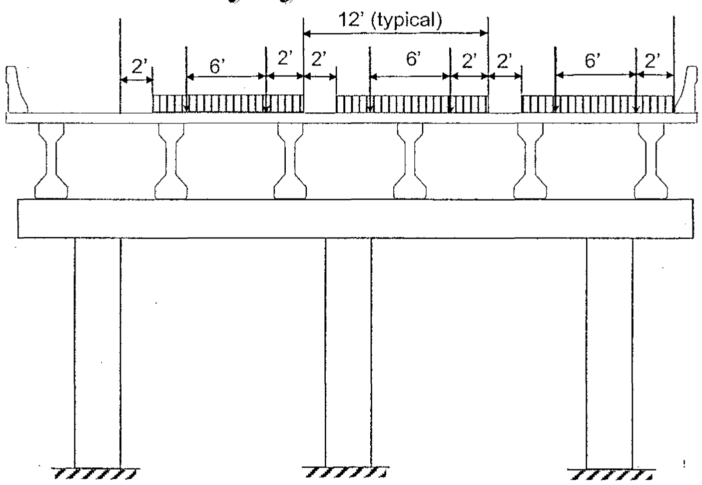
Summary of loads: Live loads



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TxDOT's Software: CAP 18

- CAP 18 origins can be trace back to 1975 (CAP 17)
- Input requires "cards"
- Win32 version (Ver 5.1) developed in 2001
- Graphical outputs are still based on a series of discrete symbols to mimic curves

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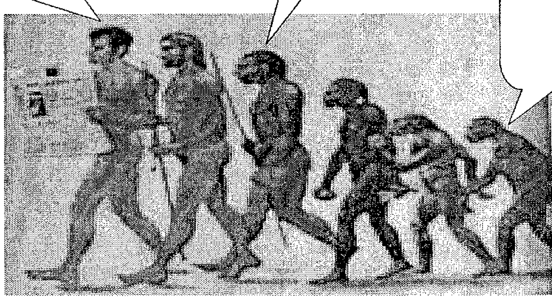
Evolution of CAP 18



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Disadvantages of CAP 18

- Old algorithm
- Beam analysis program
- Cumbersome input
- Lack of graphics
- Special treatment for one lane loading
- Modified factor for dead loads

$$\frac{\mathrm{DC}(\mathrm{LF}_{\mathrm{DC}}) + \mathrm{DW}(\mathrm{LF}_{\mathrm{DW}})}{\mathrm{DC} + \mathrm{DW}} = \mathrm{LF}_{\mathrm{CAP18}}$$

$$\frac{\mathrm{DC}(\mathrm{LF}_{\mathrm{DC}}) + \mathrm{DW}(\mathrm{LF}_{\mathrm{DW}})}{1.25} = \mathrm{DL}_{\mathrm{CAP18}}$$

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Advantages of CAP 18

- Track record of 30 years.
- TxDOT designers are familiar with its limitations.
- Limitations in analysis have been compensated through detailing.
- Apparent conservative results.
- Specifically tailored to handle moving loads in caps
- Cost

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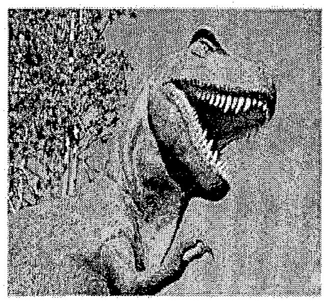
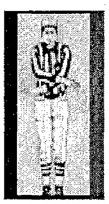
CAP 18 Summary

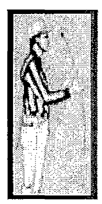
It is a dinosaur but still roars.



Future of CAP 18



- Compensate for the differences through detailing?
- Discontinue CAP 18 and get new software?



Research Methodology

Three approaches have been followed to analyze the caps:

- TxDOT's LRFD approach
- Researcher's LRFD approach
- LFD

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Salient differences between TxDOT's approach and researcher's approach

Research approach

- maintains actual axle reactions
- divides railing load between the two outermost beams
- keeps original dead load for service
- includes live load effects from both spans
- uses a constant value of 2.0 to calculate V_c

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Values of θ and β for sections with transverse reinforcement

$\frac{v}{f'_c}$	$\varepsilon_x \times 1{,}000$										
	≤-0.20	≤-0.10	≤-0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00	≤1.50	≤2.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23	40.8 1.95	43.9 1.67
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18	40.8 1.93	43.1 1.69
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.28	37.0 2.13	41.0 1.90	43.2 1.67
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08	40.5 1.82	42.8 1.61
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96	39.7 1.71	42.2 1.54
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79	39.2 1.61	41.7 1.47
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64	38.8 1.51	41.4 1.39
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50	38.6 1.38	41.2 1.29

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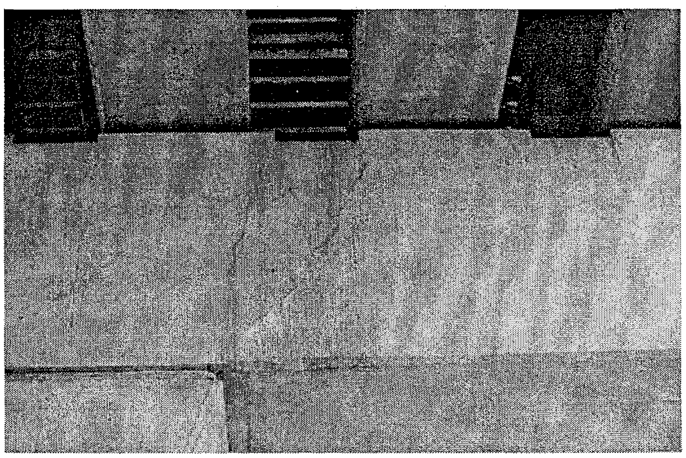
Interior Shear crack



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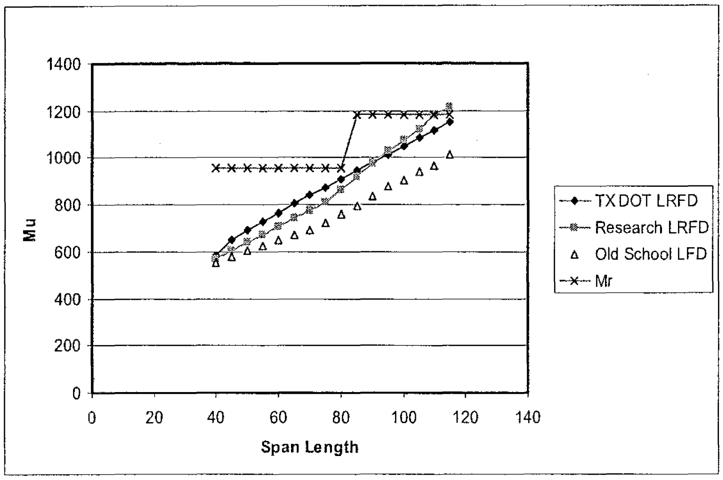
Maximum Positive Moment on 3-column bent 44’ roadway by span - 0º skew



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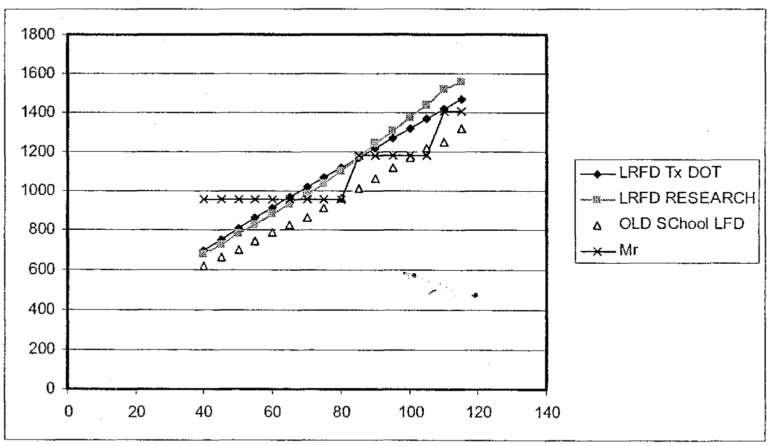
Maximum Negative Moment on 3-column bent 44’ roadway by span – 0º skew



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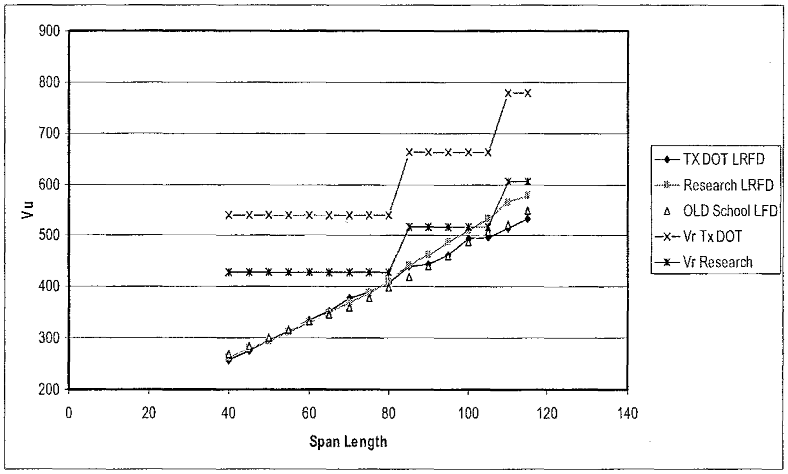
Maximum Shear on 3-column bent 44' roadway by span – 0° skew



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Concerns

- Torsion
- Two bearing lines
- Disturbed regions
- Limit on f_s
- Moments in columns

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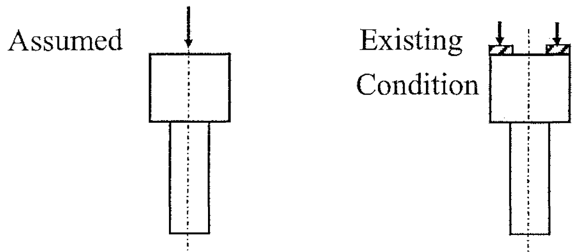
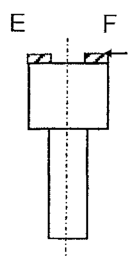
Torsion and two bearing lines



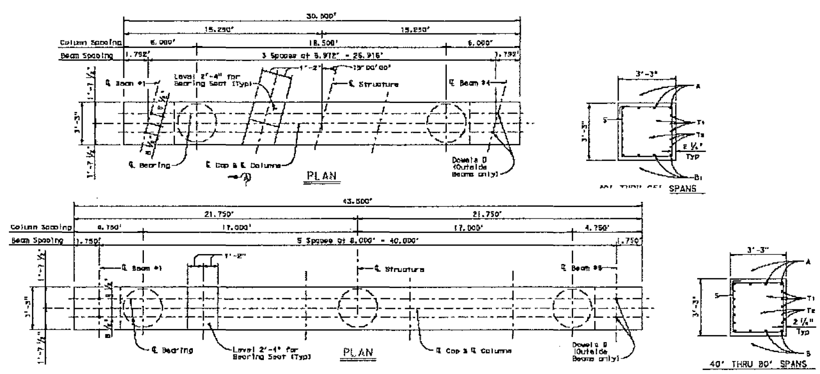
- Some agencies required a minimum eccentricity

Torsion from longitudinal forces



- Braking
- Wind on live load

Disturbed Regions



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AASHTO LRFD Article 5.8.1.1

… or components in which a load causing more than ½ of the shear at a support is closer than 2d from the face of the support, may be considered to be deep components for which the provisions of Article 5.6.3 and the detailing requirements of Article 5.13.2.3 apply.

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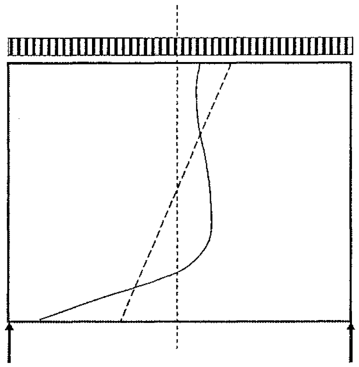
Stresses in deep beams



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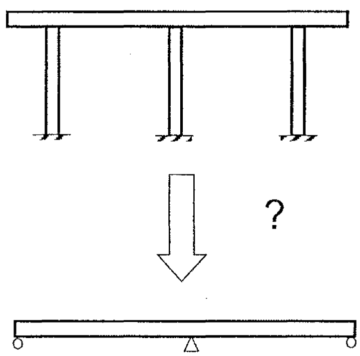
Moments in columns

Current TxDOT model does not generate any moments in the columns at the column/cap joint



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Cap design example

Span Properties

RoadwayWidth := 44 ft
OverAllwidth := 46 ft
Span := 110 ft
BeamSpace := 8 ft
NumberOfBeams := 6
BeamLength := 109.67
Skew := 0

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Cap design example (Cont'd)

Cap Dimensions

CapWidth := 3.25 ft
CapDepth := 3.25 ft
CapLength := 44 ft

Column Dimensions

ColumnDiameter= 3.0 ft
ColumnSpace:= 17.0 ft
NumberOfColumns= 3
ColumnHeight= 20 ft

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Cap design example (Cont'd)

Dead Load Constants

RailWeight := .326 klf (T-501)

BeamWeight := .821 klf (Type IV)

Overlay := 2 in

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Cap design example (Cont'd)

Reinforced Concrete Properties

fc := 3.6 ksi

fy := 60 ksi

$Ec := 33000\left[\left(0.145^{1.5}\right)\cdot\sqrt{fc}\right]$ $Ec = 3.457\times 10^{3}$ ksi

Es := 29000ksi

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Cap design example (Cont'd)

Design Lanes LRFD 3.6.1.1.1

$$\text{NoOfLanes} := \frac{\text{RoadwayWidth}}{12}$$

$$\text{trunc}(\text{NoOfLanes}) = 3$$

$$\text{MaxLanes} := \text{trunc}(\text{NoOfLanes})$$

$$\boxed{\text{MaxLanes} = 3}$$

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Cap design example (Cont'd)

Breaking Force LRFD 3.6.4

$$\text{BR1} := .25 \cdot (32 + 32 + 8) \cdot \text{MaxLanes} \cdot \text{Mpf3}$$

$\boxed{\text{BR1} = 45.9}$ kips

$$\text{BR2a} := \text{MaxLanes} \cdot \text{Mpf3} \cdot 0.05 \cdot [\, [\, 72 + (\text{Span} + \text{Span}) \cdot 0.64]\,]$$

$\boxed{\text{BR2a} = 27.132}$ kips

$$\text{BR2b} := \text{MaxLanes} \cdot \text{Mpf3} \cdot 0.05 \cdot [(25 + 25) + 2 \cdot \text{Span} \cdot .64]$$

$\boxed{\text{BR2b} = 24.327}$ kips

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Cap design example (Cont'd)

Dead Load Calculations

Rail: $DLr := RailWeight \cdot \frac{Span}{2}$

$\boxed{DLr = 17.93}$ kips/beam pair

Slab:

$ConcreteWt := .15$ kip/cf

$SlabConcrete := 130.2\ cy$

$$DLs := 27 \cdot SlabConcrete \cdot \frac{ConcreteWt \cdot 1.05}{NumberOfBeams}$$

$\boxed{DLs = 92.279}$ kips/beam pair

Beam: $DLb := BeamWeight \cdot BeamLength$

$\boxed{DLb = 90.039}$ kips/beam pair

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Cap design example (Cont'd)

Dead Load Calculations (cont'd)

Overlay: $AsphaltWt := .14$ kip/cf

$$Dwol := \frac{AsphaltWt \cdot Overlay \cdot BeamSpace \cdot Span}{12}$$

$Dwol = 20.533$ kips/beam pair

$DW := Dwol$

$\boxed{DW = 20.533}$ kips/beam pair

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Cap design example (Cont'd)

Dead Load Calculations (cont'd)

Cap:

$Station := 0.5$ ft/sta

$DLcap := CapWidth \cdot CapDepth \cdot ConcreteWt \cdot Station$

$\boxed{DLcap = 0.792}$ Cap 18 input

Station for the incremental load used in Cap 18 is set at 1/2 foot

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Cap design example (Cont'd)

Dead Load Calculations (cont'd)

Dead load total per beam pair:

$DC := DLr + DLs + DLb$

$DC = 200.248$ kips/beam pair

$$DL18F := \frac{(DC \cdot 1.25 + DLcap \cdot 1.25 + DW \cdot 1.5)}{DC + DW + DLcap}$$

$\boxed{DL18F = 1.273}$ Cap 18 factor

$DLtotal := DLr + DLs + DLb + Dwol$

$\boxed{DLtotal = 220.782}$ kip/beam Cap 18 input

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Cap design example (Cont'd)

Live load + impact calculations

$IM := 1.33$

Lane:

$LaneLoad := .64 \cdot Span$

$\boxed{LaneLoad = 70.4}$ kip

Truck: $Truck := 32 + 32\left(\frac{Span - 14}{Span}\right) + 8\cdot\left(\frac{Span - 14}{Span}\right)$

$\boxed{Truck = 66.909}$ kip

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Cap design example (Cont'd)

Live load + impact calculations

$$TruckTrain = \left[32 + 32\left(\frac{Span - 14}{Span}\right) + 8\cdot\left(\frac{Span - 28}{Span}\right) + 8\cdot\left(\frac{Span - 50}{Span}\right) + 32\left(\frac{Span - 64}{Span}\right) + 32\left(\frac{Span - 78}{Span}\right)\right]$$

$\boxed{TruckTrain = 92.945}$ kip

$ControlTruck := if(Truck \geq TruckTrain, Truck, TruckTrain)$

$\boxed{ControlTruck = 92.945}$ kip

$LLRxn := 0.9\cdot(LaneLoad + ControlTruck\cdot IM)$

$\boxed{LLRxn = 174.616}$ kip

$$P1 := \frac{(ControlTruck\cdot IM)\cdot 0.9}{2}$$

$\boxed{P1 = 55.628}$ kip Cap 18 input

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$$w := \frac{(LaneLoad)\cdot 0.90}{20}$$

$\boxed{w = 3.168}$ Cap 18 input

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Cap design example (Cont'd)

Limit States LRFD 3.4.1

DC dead load of permanent components

Dw is wearing surface components

LL is lane load plus the Truck load*1.33 impact

BR breaking force transferred from superstructure

Mr = ϕ Mn LRFD 5.5.4.2-1

bw is b

Strength 1 DC*1.25+DW*1.5+LL*1.75

=DL18F(DC+DW)+(P1+W)*1.75

Service 1 $DC \cdot 1.0 + DW \cdot 1.0 + LL \cdot 1.0$

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Cap design example (Cont'd)

Cap 18 Output (moments)

	(kip – ft) Max + M	Sta	(kip – ft) Max - M	Sta
Dead load	posDL := 365.0	70	negDL := 682.2	80
Service	posServ := 799.5	70	negServ := 1063.9	80
Ultimate	posUlt := 1181.7	70	negUlt := 1520.8	80

Max Moments

Mupos := posUlt $\quad$ $\boxed{\text{Mupos} = 1.182 \times 10^3}$ kip-ft

Muneg := negUlt $\quad$ $\boxed{\text{Muneg} = 1.521 \times 10^3}$ kip-ft

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Cap design example (Cont'd)

Minimum Flexural Reinforcement LRFD 5.7.3.3.2

$$Ig := (CapWidth \cdot 12) \cdot \frac{(CapDepth \cdot 12)^3}{12}$$

$Ig = 1.928 \times 10^5 \quad in^4$

$fr := 0.24 \cdot \sqrt{fc}$ $\qquad fr = 0.455 \quad psi$

$yt := CapDepth \cdot \frac{12}{2}$ $\qquad yt = 19.5in$

$Mcr := Ig \cdot \frac{fr}{yt}$ $\qquad Mcr = 4.502 \times 10^3$ kip-in

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Cap design example (Cont'd)

Minimum Flexural Reinforcement LRFD 5.7.3.3.2 (cont'd)

$Mcr1 := 1.2 \cdot \frac{Mcr}{12}$ $\qquad Mcr1 = 450.2$ kip ft

$Mcr2 := 1.33 \cdot posUlt$ $\qquad Mcr2 = 1.572 \times 10^3$ kip ft

$Mcr3 := 1.33 \cdot negUlt$ $\qquad Mcr3 = 2.023 \times 10^3$ kip ft

$Mfpos := if(Mcr1 \le Mcr2, Mcr1, Mcr2)$

$\boxed{Mfpos = 450.2}$

$Mfneg := if(Mcr1 \le Mcr3, Mcr1, Mcr3)$

$\boxed{Mfneg = 450.2}$

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Cap design example (Cont'd)

Moment Capacity Design LRFD 5.7.3.2

$\phi := 0.9 \quad \beta1 := 0.85$

BarNo := 6 Top

BarNoB := 5 Bottom

$As := BarNo \cdot No11 \quad As = 9.36 \quad in^2$

$AsB := BarNoB \cdot No11 \quad AsB = 7.8 \quad in^2$

$$d := (CapDepth \cdot 12) - 2 - \left(\frac{5}{8}\right) - \frac{1.41}{2} \qquad \boxed{d = 35.67}$$

$b := CapWidth \cdot 12 \quad b = 39 \quad in$

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Cap design example (Cont'd)

Moment Capacity Design (Cont'd)

$fc = 3.6 \quad ksi \qquad fy := 60 \quad ksi$

$$c := \frac{As \cdot fy}{.85 \cdot fc \cdot \beta1 \cdot b} \qquad c = 5.536 \quad in$$

$$cB := \frac{AsB \cdot fy}{.85 \cdot fc \cdot \beta1 \cdot b} \qquad cB = 4.614 \quad in$$

$a := c \cdot \beta1 \qquad a = 4.706 \quad in$

$aB := cB \cdot \beta1 \qquad aB = 3.922 \quad in$

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Cap design example (Cont'd)

Nominal Resistance

$$Mn := As \cdot fy \cdot \left(d - \frac{a}{2} \right) \qquad \boxed{Mn = 1.871 \times 10^4} \text{ kip in}$$

$$MnB := AsB \cdot fy \cdot \left(d - \frac{aB}{2} \right) \qquad \boxed{MnB = 1.578 \times 10^4} \text{ kip in}$$

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Cap design example (Cont'd)

Flexural Resistance

$$Mr := \phi \cdot \frac{Mn}{12} \qquad \boxed{Mr = 1.403 \times 10^3} \text{ kip ft}$$

$$MrB := \phi \cdot \frac{MnB}{12} \qquad \boxed{MrB = 1.183 \times 10^3} \text{ kip ft}$$

Ultimate $posUlt = 1.182 \times 10^3$ kip ft

$negUlt = 1.521 \times 10^3$ kip ft

MinFlexPos := if[(MrB ≥ Mupos), OK, NG]

MinFlexPos = "OK"

MinFlexNeg := if[(Mr ≥ Muneg), OK, NG]

MinFlexNeg = "NG"

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Cap design example (Cont'd)

LRFD 5.7.3.2

Check As Min Top

MinReinf := if[(Mr ≥ Mfneg), OK, NG]

MinReinf = "OK"

Check As Min Bottom

MinReinfB := if[(MrB ≥ Mfpos), OK, NG]

MinReinfB = "OK"

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Cap design example (Cont'd)

LRFD 5.7.3.3.1-1

Check As Top Max

$TopcdRatio := \frac{c}{d}$ TopcdRatio = 0.155

TopMaxSteel := if[(TopcdRatio ≤ 0.42), OK, NG]

TopMaxSteel = "OK"

Check As Bottom Max

$BottomcdRatio := \frac{cB}{d}$ BottomcdRatio = 0.129

BottomMaxSteel := if(BottomcdRatio ≤ 0.42, OK, NG)

BottomMaxSteel = "OK"

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Cap design example (Cont'd)

Check Serviceability Top

$dc := 2 + \left(\frac{5}{8}\right) + \frac{1.41}{2}$ $dc = 3.33$ in

$ds := dc$

$A1 := ds \cdot 2 \frac{(CapWidth \cdot 12)}{BarNo}$ $A1 = 43.29$

$z := 170$ kip/in

$fs1 := \frac{z}{\sqrt[3]{dc \cdot A1}}$ $fs1 = 32.422$ ksi

$fs2 := 0.6fy$ $fs2 = 36$ ksi

$fs := if(fs1 \le fs2, fs1, fs2)$ $\boxed{fs = 32.422}$ ksi

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Cap design example (Cont'd)

$n := \frac{Es}{Ec}$ $\boxed{n = 8.388}$

$p := \frac{As}{b \cdot d}$ $\boxed{p = 6.728 \times 10^{-3}}$

$k := -(p \cdot n) + \sqrt{(2 \cdot p \cdot n) + (p \cdot n)^2}$ $k = 0.284$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.905}$

$AllowMs := As \cdot d \cdot j \cdot \frac{fs}{12}$ $AllowMs = 816.593$ kip ft

ServiceabilityMom := if[(AllowMs ≥ negServ), OK, NG]

$\boxed{\text{ServiceabilityMom = "NG"}}$

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Cap design example (Cont'd)

Check Serviceability Bottom

$dc := 2 + \left(\frac{5}{8}\right) + \frac{1.41}{2}$ $dc = 3.33$ in

$ds := dc$

$A1B := ds \cdot 2 \frac{(CapWidth \cdot 12)}{BarNoB}$ $A1B = 51.948$

$z := 170$ kip/in

$fs1B := \frac{z}{\sqrt[3]{dc \cdot A1B}}$ $fs1B = 30.51$ ksi

$fs2 := 0.6fy$ $fs2 = 36$ ksi

$fsB := if(fs1B \le fs2, fs1B, fs2)$ $\boxed{fsB = 30.51}$ ksi

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Cap design example (Cont'd)

$n := \frac{Es}{Ec}$ $n = 8.388$

$pB := \frac{AsB}{b \cdot d}$ $pB = 5.607 \times 10^{-3}$

$kB := -(pB \cdot n) + \sqrt{(2 \cdot pB \cdot n) + (pB \cdot n)^2}$ $kB = 0.263$

$jB := 1 - \frac{kB}{3}$ $\boxed{jB = 0.912}$

$AllowMs := AsB \cdot d \cdot jB \cdot \frac{fsB}{12}$ $AllowMs = 645.318$ kip ft

$ServiceabilityMomB := if[(AllowMs \ge posServ), OK, NG]$

$\boxed{ServiceabilityMomB = \text{"NG"}}$

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Cap design example (Cont'd)

Check Dead Load Positive Moment

Check Mdl: $fdl := 22$ ksi

$AllowMdlp := AsB \cdot d \cdot jB \cdot \frac{fdl}{12}$ $AllowMdlp = 465.32$ kip ft

$DeadLoadMoment := if[(AllowMdlp \geq posDL), OK, NG]$

DeadLoadMoment = "OK"

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Cap design example (Cont'd)

Check Dead Load Negative Moment

Check Mdln: $fdl := 22$ ksi

$AllowMdln := As \cdot d \cdot j \cdot \frac{fdl}{12}$ $AllowMdln = 554.102$ kip ft

$DeadLoadMoment := if[(AllowMdln \geq negDL), OK, NG]$

DeadLoadMoment = "NG"

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Cap design example (Cont'd)

Skin Reinforcment 5.7.3.4

$NoOfSkinBars := 5 \quad AreaNo5 := .31 \quad Dia5 := .625 \text{ in}$

$Ask := NoOfSkinBars \cdot AreaNo5 \quad Ask = 1.55 \text{ in}^2$

$Cover := 2.25 \text{ in} \quad Dia11 := 1.41 \text{ in}$

$AskMin := 0.012(d - 30) \quad AskMin = 0.068 \text{ in}^2$

$TensionSteel := if(As \geq AsB, As, AsB)$

$TensionSteel = 9.36$

$$MinSkin := if\left(Ask \leq \frac{TensionSteel}{4}, OK, NG\right)$$

$\boxed{MinSkin = \text{"OK"}}$

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Cap design example (Cont'd)

$de := d \quad de = 35.67 \text{ in}$

$$MaxSkSp := \frac{de}{6} \quad MaxSkSp = 5.945 \text{ in}$$

Check Max spacing de/6 or 12 in

$$SkinSpProv := \frac{[(CapDepth \cdot 12) - (Cover \cdot 2 + Dia5 \cdot 2 + Dia11 \cdot 2)]}{NoOfSkinBars + 1}$$

$SkinSpProv = 5.072 \text{ in}$

$SkinSpace := if[(SkinSpProv \leq MaxSkSp), OK, NG]$

$\boxed{SkinSpace = \text{"OK"}}$

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Cap design example (Cont'd)

Shear Design (LRFD 5.8)

Flow Chart design procedure see Figure C 5.8.3.4.2-5

$\beta := 2$

$\theta := 45$

$\phi v := 0.90$

$bv := b$

$bv = 39$ in

$Vs = (A_v * f_y * d_v * (\cot\theta + \cot\alpha) * \sin\alpha) * 1/S$

For $\theta = 45$ and $\alpha = 90$ it reduces to

$$Vs = A_v d_v f_y / S$$

$Vp := 0$ Prestress

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Cap design example (Cont'd)

Cantilever Section

$Vu := 520.0$ kips Sta 81 From Cap 18 output

$Sp := 6.0$ in $Av := .62$ in^2

$Mn = 1.871 \times 10^4$ kip in with BarNo = 6

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Cap design example (Cont'd)

Finding dv at cantilever section

$dv1 := \frac{Mn}{As \cdot fy}$ $dv1 = 33.317$ in

$dv2 := 0.9d$ $dv2 = 32.103$ in

$dv3 := 0.72 \cdot (CapDepth \cdot 12)$ $dv3 = 28.08$ in

$tempdv := if[(dv2 \geq dv3), dv1, dv3]$

$dv := if(dv1 \geq tempdv, dv1, tempdv)$

$dv = 33.317$ in

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Cap design example (Cont'd)

$Vc := 0.0316 \cdot \beta \sqrt{fc} \cdot bv \cdot dv$ $Vc = 155.812$ kips

$Vnmax := 0.25 \cdot fc \cdot bv \cdot dv$

$Vnmax = 1.169 \times 10^{3}$ kips

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Cap design example (Cont'd)

$$Vs := Av \cdot dv \cdot \frac{fy}{Sp} \qquad Vs = 206.566 \text{ kips}$$

$$Vn := Vc + Vs \qquad Vn = 362.377 \text{ kips}$$

$$Vn := \text{if}[(Vnmax \le Vn), Vnmax, Vn]$$

$$Vn = 362.377 \text{ kips}$$

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Cap design example (Cont'd)

$$Vr := \phi v \cdot Vn \qquad Vr = 326.14 \text{ kips}$$

$$v := \frac{Vu - \phi v \cdot Vp}{\phi v \cdot b \cdot dv} \qquad v = 0.445 \text{ ksi}$$

MaxVrCL:= if[(Vr ≥ Vu), OK, NG]

MaxVrCL= "NG"

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Transitioning to LRFD Design Issues and Recommendations

Mary Beth Hueste

Dennis Mertz

Texas Transportation Institute

Partial Debonding of Strands

- LRFD
  - Debonding limited to 40% per row and 25% per section
  - The use of greater percentages of partially debonded strands is allowed based on the successful past practices.
- Based on research by Barnes, Burns and Kreger (1999) and successful past practice by TxDOT:
  - *Up to 75% of the strands may be debonded, if*
    - Cracking is prevented in or near the transfer length
    - AASHTO LRFD rules for terminating the tensile reinforcement are applied to the bonded length of prestressing strands.
    - The shear resistance at the regions where the strands are debonded should be thoroughly investigated with due regard to the reduction in horizontal force available, as recommended in LRFD commentary 5.11.4.3

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LRFD Distribution Factors

Number of Beams (N_b) Limitation

- For uniform distribution of permanent dead loads, the number of beams (N_b) ≥ 4 [LRFD Art. 4.6.2.2]
- Parametric study: limitation violated for the selected U54 girder spacings of 14 ft. and 16.67 ft. where $N_b = 3$.
- Need for refined analysis methods to determine actual distribution of permanent dead loads (relatively uneconomical and time consuming)

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LRFD Distribution Factors

Edge Distance Parameter (d_e) Limitation

- $d_e \leq 3.0\, ft.$ is a very restrictive limit

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LRFD Distribution Factors

Span Length Limitation

- Use of the LRFD live load DFs is limited to spans no longer than 140 ft. for spread box beams
- Slightly violated for the 8.5 ft. girder spacing with a 60 degree skew (corresponding maximum span = 144 ft.).

Parameters for Refined Analysis – U54 Girders

Span (ft.)	Spacing (ft.)	Skew (degrees)	Total Number of Cases	LRFD Restrictions
140, 150	8.5	60	2	$L \leq 140$ ft. *

* This restriction is related to the LRFD Live Load DF formulas to be applicable.

- Investigated using *grillage* analysis
- LRFD live load DFs found applicable for span lengths of 140 and 150 ft.

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LRFD Distribution Factors

- Further study is recommended to develop new, or verify the current, formulas for load distribution beyond the limitations given in LRFD Art. 4.6.2.2.

<u>Cases to Consider</u>

- – Number of beams equal to 3
- – Edge distance parameter value greater than 3.0 ft.
- – Span length greater than 140 ft.
- – Girder spacing greater than 11.5 ft. (for shear correction factor for skew)

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Transverse Shear Design

- Issues
  - Complex design procedure using MCFT.
  - MCFT being a relatively new theory, bridge engineers might face some problems employing it in practice.

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Transverse Shear Design

- Recommendations
  - Educate bridge engineers about MCFT applications
  - Find simplified design procedures applicable to typical bridges (ie. UIUC research)

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Interface Shear

- Issues
  - Significant increase in the required area of horizontal shear reinforcement
  - TxDOT currently does not let horizontal shear to govern the transverse design and uses the reinforcement from transverse shear design for horizontal shear.
  - For almost all the girders in parametric study, designed using LRFD Specifications, horizontal shear governs the design of transverse reinforcement.

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Interface Shear

- Present LRFD provisions are extremely conservative as compared to existing literature and experimental data.
- LRFD provisions are based on pure shear friction model which assumes interface shear proportional to the clamping force of the reinforcement crossing the interface.

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Interface Shear

- <u>Recommendations for Transitioning to LRFD</u>
  - A decision on whether to keep using the same methodology can be taken based on the past experience of the interface shear performance of the bridge girders not designed using TxDOT methodology.
  - Perform experimental research to find the interface shear performance of typical prestressed bridge girders
  - AASHTO is proposing to revise the interface shear provisions, already approved by T-10 committee.
  - The proposed provisions can be used once approved by AASHTO

Interface Shear – T-10 Committee

- The proposed horizontal shear provisions are provided as follows:

The nominal shear resistance of the interface surface is given as:

$$V_{ri} = \phi V_{ni}$$
$$V_{ni} \leq V_{ui}$$

V_{ni} = Nominal interface shear resistance, kips
V_{ui} = Factored interface shear force due to total load based on strength load combinations, kips
ϕ = Resistance factor for shear = 0.9

Interface Shear – T-10 Committee

Case I: Interface of concrete girders with top surface roughened to an amplitude of 0.25 in. and cast-in-place concrete slab

$$V_{ni} = (0.28 + \rho_{vi} f_y) A_{cv}$$

$$V_{ni} \leq 0.3 f_c' A_{cv} \text{ or}$$

$$V_{ni} \leq K_i A_{cv}$$

ρ_{vi} = Interface reinforcement ratio = A_{vf}/A_c

A_{cv} = Area of concrete considered engaged in shear transfer, in^2 = $b_{vi} d_v$

A_{vf} = Area of interface shear reinforcement crossing the shear plane within A_{cv}, in^2

b_{vi} = Interface width considered engaged in shear transfer

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Interface Shear – T-10 Committee

d_v = Effective shear depth, in.

f_y = Yield strength of reinforcement, ksi

f_c' = Specified design strength of weaker concrete on either side of the interface

K_i = 1.6 for normal weight concrete

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Interface Shear – T-10 Committee

2) Generalized interface shear applications (similar format as LRFD)

$$V_{ni} = [c + \mu\,(\rho_{vi} f_y)]\,A_{cv}$$
$$V_{ni} \leq 0.2 f_c' A_{cv} \text{ or}$$
$$V_n \leq 0.8 A_{cv}$$

c = Cohesion factor
μ = Friction factor
P_c = Permanent net compressive force normal to the shear plane, kips

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Interface Shear – T-10 Committee

For concrete placed against clean, hardened concrete and free of laitance, but and intentionally roughened surface to an amplitude of 0.25 in.:

$c = 0.280$ ksi (compared to 0.10 ksi in present LRFD)
$\mu = 1.0$ (same as LRFD)

For concrete placed against clean, hardened concrete and free of laitance, but not an intentionally roughened surface:

$c = 0.075$ ksi (same as LRFD)
$\mu = 0.6$ (same as LRFD)

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Interface Shear – T-10 Committee

- Minimum $A_{vf} \geq (0.05A_{cv})/f_y$
- $V_{ni} \geq 1.33\ V_{ui}$

Interface Shear – T-10 Committee

- Proposed Provisions
  - Uses modified shear friction model, which yields results comparable to experimental results and past experience.
  - Modified shear friction model considers the combined resistance of cohesion and friction mobilized by interface reinforcement clamping force.
  - Recommends use of ACI 318-02 and AASHTO Standard Specifications method to calculate horizontal interface shear requirements.
  - Same as the present provisions with changes in cohesion factors.
  - Yields results close to that of Standard Specifications.

Evolving Issues

- Lateral Live-Load Distribution
- Structural Concrete Shear Design
- Structural Concrete Interface-Shear Design
- Deep-Foundation Design
- Fatigue Design
- Load and Resistance Factor Rating (LRFR)

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Concluding Remarks
Evaluation Forms
CEU Certificates

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Appendix A

Detailed Examples for Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design

DRAFT

August 29, 2005

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Table of Contents

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A.2 Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design using AASHTO LRFD Specifications

A.2.1 INTRODUCTION

Following is a detailed example showing sample calculations for the design of a typical interior AASHTO Type IV prestressed concrete girder supporting a single span bridge. The design is based on the *AASHTO LRFD Bridge Design Specifications 3rd Edition, 2004* (AASHTO 2004). The recommendations provided by the TxDOT Bridge Design Manual (TxDOT 2001) are considered in the design. The number of strands and concrete strength at release and at service are optimized using the TxDOT methodology.

A.2.2 DESIGN PARAMETERS

The bridge considered for this design example has a span length of 110 ft. (c/c pier distance), a total width of 46 ft. and total roadway width of 44 ft. The bridge superstructure consists of six AASHTO Type IV girders spaced 8 ft. center-to-center, designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck. The wearing surface thickness is 1.5 in., which includes the thickness of any future wearing surface. T501 type rails are considered in the design. HL-93 is the design live load. A relative humidity (RH) of 60% is considered in the design and the skew angle is 0 degrees. The bridge cross section is shown in Figure A.2.2.1.

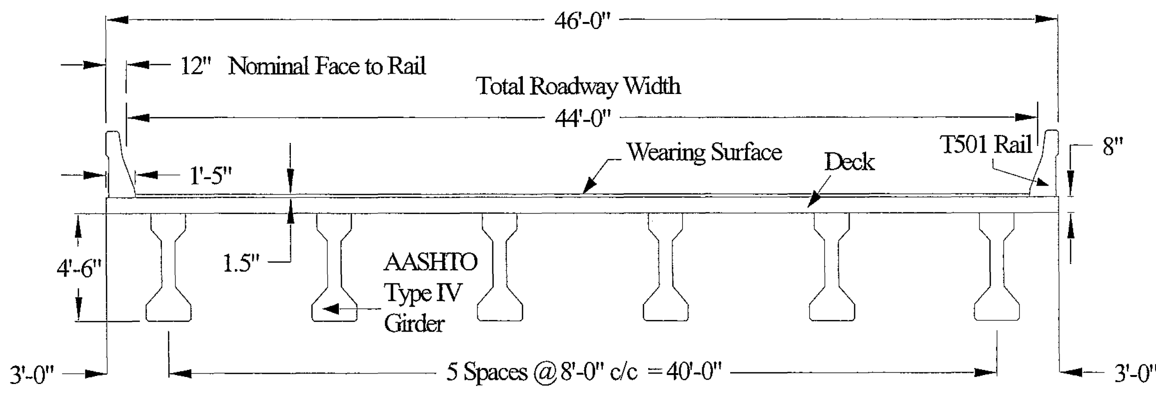

Figure A.2.2.1 Bridge Cross Section

The design span and the overall girder length are based on the following calculations.

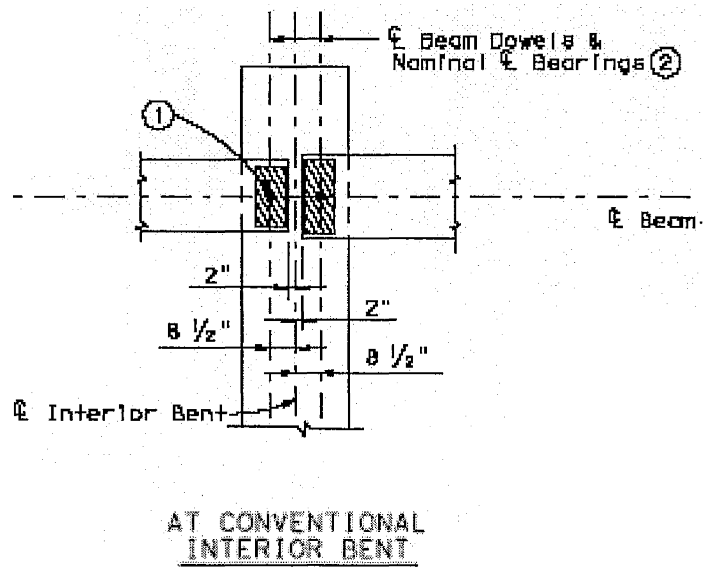

Figure A.2.2.2. Girder End Details (TxDOT Standard Drawing 2001)

Span Length (c/c piers) = 110'-0"
From Figure A.2.2.2

Overall girder length = 110 ft. – 2(2 in.) = 109'-8"

Design Span = 110 ft. – 2(8.5 in.)
= 108'-7" = 108.583 ft. (center-to-center of bearing)

A.2.3 MATERIAL PROPERTIES

Cast-in-place (CIP) slab:
Thickness, t_s = 8.0 in.
Concrete Strength at 28-days, f'_c = 4,000 psi

Thickness of asphalt wearing surface (including any future wearing surface), t_w = 1.5 in.

Unit weight of concrete, w_c = 150 pcf

Precast girders: AASHTO Type IV
Concrete Strength at release, f'_{ci} = 4,000 psi (This value is taken as an initial guess and will be finalized based on optimum design.)

Concrete Strength at 28 days, f'_c = 5,000 psi (This value is taken as initial guess and will be finalized based on optimum design)

Concrete unit weight, w_c = 150 pcf

Pretensioning strands: ½ in. diameter, seven wire low relaxation

Area of one strand = 0.153 in.^{2}

Ultimate stress, f_{pu} = 270,000 psi

Yield strength, $f_{py} = 0.9 f_{pu}$ = 243,000 psi

[LRFD Table 5.4.4.1-1]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

Before transfer, $f_{pi} \leq 0.75 f_{pu}$ = 202,500 psi

At service limit state (after all losses)
$f_{pe} \leq 0.80 f_{py}$ = 194,400 psi

Modulus of Elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Nonprestressed reinforcement:

Yield strength, f_y = 60,000 psi

Modulus of Elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Unit weight of asphalt wearing surface = 140 pcf

[TxDOT recommendation]

T501 type barrier weight = 326 plf /side

A.2.4 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR GIRDER

A.2.4.1 Non-Composite Section

The section properties of an AASHTO Type IV girder as described in the TxDOT Bridge Design Manual (TxDOT 2001) are provided in Table A.2.4.1. The section geometry and strand pattern are shown in Figures A.2.4.1 and A.2.4.2, respectively.

Table A.2.4.1 Section Properties of AASHTO Type IV girder (notations as used in Figure A.2.4.1, Adapted from TxDOT Bridge Design Manual (TxDOT 2001))

A	B	C	D	E	F	G	H	W	y_t	y_b	Area	I	Wt./lf
in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.^{2}	in.^{4}	Lbs
20	26	8	54	9	23	6	8	8	29.25	24.75	788.4	260,403	821

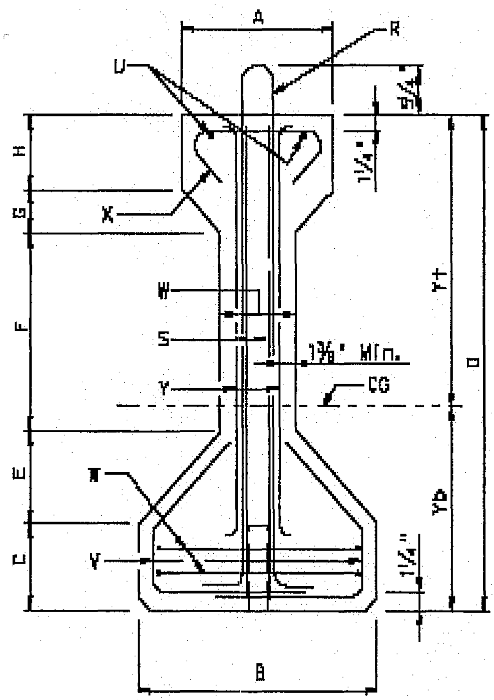

Figure A.2.4.1 Section Geometry of AASHTO Type IV Girder (TxDOT 2001)

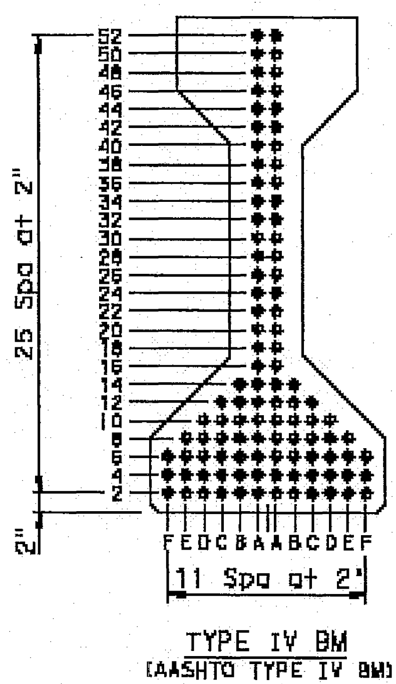

Figure A.2.4.2 Strand Pattern for AASHTO Type IV Girder (TxDOT 2001)

I = Moment of inertia about the centroid of the non-composite precast girder = 260,403 in.^{4}

y_b = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

y_t = Distance from centroid to the extreme top fiber of the non-composite precast girder = 29.25 in.

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.^{3}
= I/y_b = 260,403/24.75 = 10,521.33 in.^{3}

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder, in.^{3}
= I/y_t = 260,403/29.25 = 8,902.67 in.^{3}

A.2.4.2 Composite Section

A.2.4.2.1 Effective Flange Width

The effective flange width is lesser of: [LRFD Art. 4.6.2.6.1]

1/4 span length: $\frac{108.583(12\text{ in./ft.})}{4} = 325.75$ in.

12(effective slab thickness) + greater of web thickness or ½ girder top flange width: 12(8) + ½ (20) = 106 in.
(½(girder top flange width) = 10 in. > web thickness = 8 in.)

Average spacing of adjacent girders: 8(12 in./ft.) = 96 in. (controls)

Effective flange width = 96 in.

A.2.4.2.2 Modular Ratio Between Slab and Girder Concrete

Following the TxDOT Bridge Design Manual (TxDOT 2001) recommendation (Pg. #7-85), the modular ratio between slab and girder concrete is taken as 1. This assumption is used for service load design calculations. For flexural strength limit design, shear design and deflection calculations the actual modular ratio based on optimized concrete strengths is used.

$$n = \left(\frac{E_c \text{ for slab}}{E_c \text{ for girder}}\right) = 1$$

where n is the modular ratio between slab and girder concrete and E_c is the elastic modulus of concrete.

A.2.4.2.3 Transformed Section Properties

Transformed flange width = n(effective flange width)
= 1(96) = 96 in.

Transformed Flange Area = n(effective flange width) (t_s)
= 1(96)(8) = 768 in.^{2}

Table A.2.4.2 Properties of Composite Section

	Transformed Area A (in.^{2})	y_b in.	$A\,y_b$ in.	$A(y_{bc} - y_b)^2$	I in.^{4}	$I + A(y_{bc} - y_b)^2$ in.^{4}
Girder	788.4	24.75	19,512.9	212,231.53	260,403.0	472,634.5
Slab	768.0	58.00	44,544.0	217,868.93	4,096.0	221,964.9
$\sum$	1,556.4		64,056.9			694,599.5

A_c = Total area of composite section = 1,556.4 in.^{2}

h_c = Total height of composite section = 54 + 8 = 62 in.

I_c = Moment of inertia about the centroid of the composite section = 694,599.5 in.^{4}

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.
= 64056.9/1556.4 = 41.157 in.

y_{tg} = Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.
= 54 - 41.157 = 12.843 in.

y_{tc} = Distance from the centroid of the composite section to extreme top fiber of the slab = 62 - 41.157 = 20.843 in.

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.^{3}
= I_c/y_{bc} = 694,599.5/41.157 = 16,876.83 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder, in.^{3}
= I_c/y_{tg} = 694,599.5/12.843 = 54,083.9 in.^{3}

S_{tc} = Section modulus of composite section referenced to the top fiber of the slab, in.^{3}

$$= \left(\frac{1}{n}\right) I_c/y_{tc} = 1(694{,}599.5/20.843) = 33{,}325.31 \text{ in.}^3$$

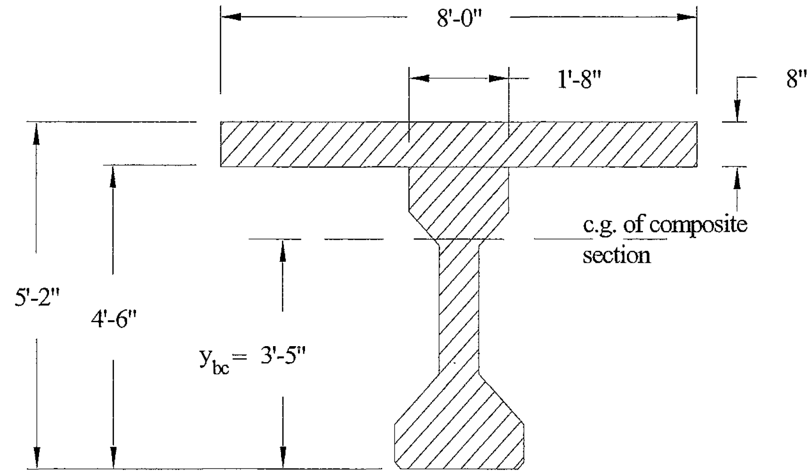

Figure A.2.4.3 Composite Section

A.2.5 SHEAR FORCES AND BENDING MOMENTS

The self-weight of the girder and the weight of slab act on the non-composite simple span structure, while the weight of the barriers, future wearing surface, live load and dynamic load act on the composite simple span structure.

A.2.5.1 Shear Forces and Bending Moments due to Dead Loads

A.2.5.1.1 Dead Loads

[LRFD Art. 3.3.2]

Dead loads acting on the non-composite structure:

Self-weight of the girder = 0.821 kip/ft.
(TxDOT Bridge Design Manual (TxDOT 2001))

Weight of cast in place deck on each interior girder

$$= (0.150 \text{ kcf})\left(\frac{8 \text{ in.}}{12 \text{ in./ft.}}\right)(8 \text{ ft.}) = 0.800 \text{ kips/ft.}$$

Total dead load on non-composite section
= 0.821 + 0.800 = 1.621 kips/ft.

A.2.5.1.2 Superimposed Dead Load

Dead loads placed on the composite structure: The permanent loads on the bridge including loads from railing and wearing surface can be distributed uniformly among all girders given the following conditions are met. [LRFD Art. 4.6.2.2.1]

1. Width of deck is constant (O.K.)

2. Number of girders, N_b is not less than four
   Number of girders in present case, $N_b = 6$ (O.K.)

3. Girders are parallel and have approximately the same stiffness (O.K.)

4. The roadway part of the overhang, $d_e \leq 3.0$ ft.
   where d_e is the distance from exterior web of the exterior girder to the interior edge of curb or traffic barrier, ft.

   d_e = (overhang distance from the center of exterior girder to bridge end) – (½ web width) – (width of barrier)
   = 3.0 – 0.33 - 1.0 = 1.67 ft. < 3.0 ft. (O.K.)

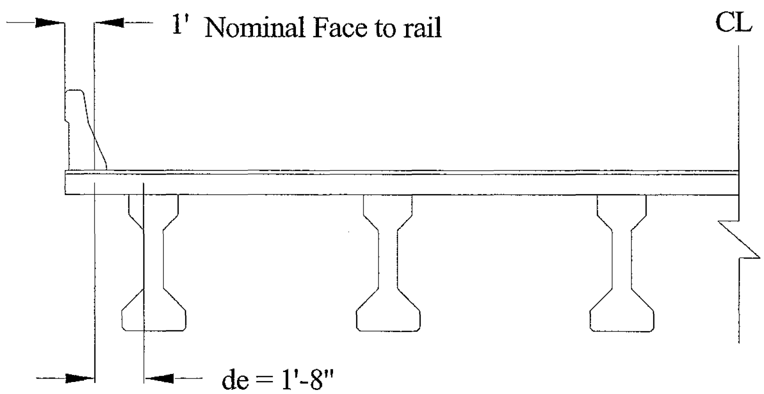

Figure A.2.5.1 Illustration of d_e calculation

5. Curvature in plan is less than 4^0 (curvature = 0^0) (O.K.)

6. Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.2.1-1
   Precast concrete I sections are specified as Type k (O.K.)

Since all the above criteria are satisfied, the barrier and wearing surface loads are equally distributed among the 6 girders.

Weight of T501 rails or barriers on each girder

$$= 2\left(\frac{326 \text{ plf}/1000}{6 \text{ girders}}\right) = 0.109 \text{ kips/ft./girder}$$

Weight of 1.5" wearing surface

$= (0.140 \text{ kcf})\left(\dfrac{1.5 \text{ in.}}{12 \text{ in/ft}}\right) = 0.0175$ kips/ft. This load is applied over the entire clear roadway width of 44'-0"

Weight of wearing surface on each girder

$$= \frac{(0.0175 \text{ ksf})(44.0 \text{ ft.})}{6 \text{ girders}} = 0.128 \text{ kips/ft./girder}$$

Total superimposed dead load = 0.109 + 0.128 = 0.237 kip/ft./girder

A.2.5.1.3 Unfactored Shear Forces and Bending Moments

Shear forces and bending moments for the girder due to dead loads, superimposed dead loads at every tenth of the design span, and at critical sections (hold down point or harp point and critical section for shear) are provided in this section. The bending moment (M) and shear force (V) due to uniform dead loads and uniform superimposed dead loads at any section at a distance x from the center line of bearing are calculated using the following formulas, where the uniform load is denoted as w.

$$M = 0.5w\,x\,(L - x)$$

$$V = w(0.5L - x)$$

The distance of critical section for shear from the support is calculated using an iterative process illustrated in the shear design section. As an initial guess the critical section for shear is taken as $(h_c/2)$ + ½ (bearing width) = (62/2) + (7/2) = 34.5 in. = 2.875 ft. from the centerline of bearing

As per the recommendations of the TxDOT Bridge Design Manual (Chap. 7, Sec 21), the distance of the hold down point (HD) from the centerline of bearing is taken as lesser of:

(½ span length – span length/20) or (½ span length – 5 ft.)

$$\frac{108.583}{2} - \frac{108.583}{20} = 48.862 \text{ ft. or } \frac{108.583}{2} - 5 = 49.29 \text{ ft.}$$

HD = 48.862 ft.

The shear forces and bending moments due to dead loads and superimposed loads are shown in Tables A.2.5.1 and A.2.5.2 respectively.

Table A.2.5.1. Shear forces due to Dead and Superimposed Dead Loads

Distance form bearing centerline x	Section x/L	Dead Loads		Super Imposed Dead Loads			Total Dead Load
		Girder weight	Slab weight	Barrier weight	Wearing surface weigh	Total	
ft.		kips	kips	kips	kips	kips	kips
0.000	0.000	44.57	43.43	5.92	6.95	12.87	100.87
2.875	0.026	42.21	41.13	5.60	6.58	12.19	95.53
10.858	0.100	35.66	34.75	4.73	5.56	10.29	80.70
21.717	0.200	26.74	26.06	3.55	4.17	7.72	60.52
32.575	0.300	17.83	17.37	2.37	2.78	5.15	40.35
43.433	0.400	8.91	8.69	1.18	1.39	2.57	20.17
48.862	0.450 (*HD*)	4.46	4.34	0.59	0.69	1.29	10.09
54.292	0.500	0.00	0.00	0.00	0.00	0.00	0.00

Table A.2.5.2. Bending Moments due to Dead and Superimposed Dead Loads

Distance form bearing centerline x	Section x/L	Dead Loads		Super Imposed Dead Loads			Total Dead Load
		Girder weight	Slab weight	Barrier weight	Wearing surface weight	Total	
ft.		k-ft.	k-ft.	k-ft.	k-ft.	k-ft.	k-ft.
0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00
2.875	0.026	124.76	121.56	16.56	19.45	36.01	282.33
10.858	0.100	435.58	424.44	57.83	67.91	125.74	985.76
21.717	0.200	774.40	754.59	102.81	120.73	223.55	1752.54
32.575	0.300	1016.38	990.38	134.94	158.46	293.40	2300.16
43.433	0.400	1161.58	1131.86	154.22	181.10	335.31	2628.75
48.862	0.450 (*HD*)	1197.87	1167.24	159.04	186.76	345.79	2710.90
54.292	0.500	1209.98	1179.03	160.64	188.64	349.29	2738.30

A.2.5.2 Shear Forces and Bending Moments due to Live Load

A.2.5.2.1 Live Load

[LRFD Art. 3.6.1.2]

The LRFD Specifications specify a significantly different live load as compared to the Standard Specifications. The LRFD design live load is designated as HL-93 which consists of a combination of the:

- Design truck with dynamic allowance or design tandem with dynamic allowance, whichever produces greater moments and shears, and

- Design lane load without dynamic allowance.

[LRFD Art. 3.6.1.2.2]

The design truck is designated as HS 20 consisting of an 8 kip front axle and two 32 kip rear axles.

[LRFD Art. 3.6.1.2.3]

The design tandem consists of a pair of 25.0-kip axles spaced 4.0 ft. apart. However, for spans longer than 40 ft. the tandem loading does not govern, thus only the truck load is investigated in this example.

[LRFD Art. 3.6.1.2.4]

The lane load consists of a load of 0.64 klf uniformly distributed in the longitudinal direction.

A.2.5.2.2 Live Load Distribution Factor for a Typical Interior Girder

The distribution factors specified by LRFD Specifications have changed significantly as compared to the Standard Specifications which specifies $S/11$ (S is the girder spacing) to be used as the distribution factor.

[LRFD Art. 4.6.2.2]

The bending moments and shear forces due to live load can be distributed to individual girders using simplified approximate distribution factors specified by LRFD Specifications. However the simplified live load distribution factors can be used only if the following conditions must be met:

[LRFD Art. 4.6.2.2.1]

1. Width of deck is constant (O.K.)

2. Number of girders, N_b is not less than four
   Number of girders in present case, $N_b = 6$ (O.K.)

3. Girders are parallel and have approximately the same stiffness (O.K.)

4. The roadway part of the overhang, $d_e \leq 3.0$ ft.
   where d_e is the distance from exterior web of the exterior girder to the interior edge of curb or traffic barrier, ft.

   d_e = (overhang distance from the center of exterior girder to bridge end) – (½ web width) – (width of barrier)
   = 3.0 – 0.33 - 1.0 = 1.67 ft. < 3.0 ft. (O.K.)

5. Curvature in plan is less than 4^0 (curvature = 0^0) (O.K.)

6. Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.2.1-1
   Precast concrete I sections are specified as Type k (O.K.)

The number of design lanes is computed as follows:

Number of design lanes = Integer part of the ratio $w/12$

where w is the clear roadway width between the curbs = 44 ft.
[LRFD Art. 3.6.1.1.1]

Number of design lanes = Integer part of (44/12) = 3 lanes.

A.2.5.2.2.1 Distribution factor for Bending Moment

The approximate distribution factors for distribution of live loads per lane for moment in interior girders are specified by LRFD Table 4.6.2.2.2b-1. The distribution factors for type k (prestressed concrete I section) bridges can be used if the following additional requirements are satisfied:

$3.5 \leq S \leq 16$, where S is the spacing between adjacent girders, ft.
S = 8.0 ft (O.K.)

$4.5 \leq t_s \leq 12$, where t_s is the slab thickness, in.
t_s = 8.0 in (O.K.)

$20 \leq L \leq 240$, where L is the design span length, ft.
L = 108.583 ft. (O.K.)

$N_b \geq 4$, where N_b is the number of girders in the cross section.
N_b = 6 (O.K.)

$10{,}000 \leq K_g \leq 7{,}000{,}000$ where K_g is the longitudinal stiffness parameter, in.^{4}

$K_g = n(I + A\, e_g^2)$ [LRFD Art. 3.6.1.1.1]

where:

n = Modular ratio between girder and slab concrete.

$= \dfrac{E_c \text{ for girder concrete}}{E_c \text{ for deck concrete}} = 1$

Note that this ratio is the inverse of the one defined for composite section properties in Section A.2.4.2.2.

A = Area of girder cross section (non-composite section) = 788.4 in.^{2}

I = Moment of inertia about the centroid of the noncomposite precast girder = 260403 in.^{4}

e_g = Distance between centers of gravity of the girder and slab, in.
= $(t_s/2 + y_t)$ = (8/2 + 29.25) = 33.25 in.

$K_g = 1[260403 + 788.4\,(33.25)^2] = 1{,}132{,}028.5$ in.^{4} (O.K.)

The approximate distribution factors for distribution of live loads per lane for moment in interior girders specified by LRFD Specifications are applicable in this case as all the requirements are satisfied. Table 4.6.2.2.2b-1 specifies the distribution factor for all limit states except fatigue limit state for interior girders of type k bridges as follows:

For one design lane loaded:

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0\,L\,t_s^3}\right)^{0.1}$$

where:

DFM = Distribution factor for live load per lane for moment in interior girders.

S = Spacing of adjacent girders = 8 ft.

L = Design span length = 108.583 ft.

t_s = Thickness of slab = 8 in.

$$DFM = 0.06 + \left(\frac{8}{14}\right)^{0.4}\left(\frac{8}{108.583}\right)^{0.3}\left(\frac{1{,}132{,}028.5}{12.0(108.583)(8)^3}\right)^{0.1}$$

$DFM = 0.06 + (0.8)(0.457)(1.054) = 0.445$ lanes/girder

For two or more lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,L\,t_s^3}\right)^{0.1}$$

$$DFM = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{108.583}\right)^{0.2}\left(\frac{1{,}132{,}028.5}{12.0(108.583)(8)^3}\right)^{0.1}$$

$$= 0.075 + (0.902)(0.593)(1.054) = 0.639 \text{ lanes/girder}$$

The greater of the distribution factor for single lane loaded and multiple lanes loaded governs. Thus the case of two or more lanes loaded controls in this case.

$DFM = 0.639$ lanes/girder

A.2.5.2.2.2 Distribution factor for Shear Force

The approximate distribution factors for distribution of live loads per lane for shear in interior girders are specified by LRFD Table 4.6.2.2.3a-1. The distribution factors for type k (prestressed concrete I section) bridges can be used if the following requirements are satisfied:

$3.5 \leq S \leq 16$, where S is the spacing between adjacent girders, ft.
$S = 8.0$ ft. (O.K.)

$4.5 \leq t_s \leq 12$, where t_s is the slab thickness, in.
$t_s = 8.0$ in (O.K.)

$20 \leq L \leq 240$, where L is the design span length, ft.
$L = 108.583$ ft. (O.K.)

$N_b \geq 4$, where N_b is the number of girders in the cross section.
$N_b = 6$ (O.K.)

The approximate distribution factors for distribution of live loads per lane for shear in interior girders specified by LRFD Specifications are applicable in this case as all the requirements are satisfied. Table 4.6.2.2.3a-1 specifies the distribution factor for all limit states for interior girders of type k bridges presented as follows.

For one design lane loaded:

$$DFV = 0.36 + \left(\frac{S}{25.0}\right)$$

where:

DFV = Distribution factor for live load per lane for shear in interior girders.

S = Girder spacing = 8 ft.

$$DFV = 0.36 + \left(\frac{8}{25.0}\right) = 0.68 \text{ lanes/girder}$$

For two or more lanes loaded:

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2$$

$$DFV = 0.2 + \frac{8}{12} - \left(\frac{8}{35}\right)^2 = 0.814 \text{ lanes/girder}$$

The greater of the distribution factor for single lane loaded and multiple lanes loaded governs. Thus the case of two or more lanes loaded controls in this case.

DFV = 0.814 lanes/girder

The distribution factor for the distribution of moments and shears in the design using Standard Specifications was 0.727 lanes/girder.

A.2.5.2.2.3 Skew Reduction

LRFD Article 4.6.2.2.2e specifies the skew reduction for load distribution factors for moment in longitudinal beams on skewed supports. The LRFD Table 4.6.2.2.2e-1 presents the skew reduction formulas for type k bridges skewed such that skew angle θ is such that $30° \leq \theta \leq 60°$.

For type k bridges having skew angle such that $\theta < 30°$, the skew reduction is zero and for skew angles $\theta > 60°$, the skew reduction is same as for $\theta = 60°$. The distribution factors for shear need not be reduced for skew.

For the present design skew angle is 0°, thus the skew reduction for load distribution factors for moment is not required.

A.2.5.2.3
Dynamic Allowance

The LRFD Specifications specify the dynamic load effects as a percentage of the static live load effects. LRFD Table 3.6.2.1-1 specifies the dynamic allowance to be taken as 33% of the static load effects for all limit states except fatigue limit state and 15% for fatigue limit state. The factor to be applied to the static load shall be taken as:

$$(1 + IM/100)$$

where

IM = Dynamic load allowance, applied to truck load or tandem load only
= 33% for all limit states except fatigue limit state.
= 15% for fatigue limit state.

The Standard Specifications specifies the impact factor to be calculated using the following equation

$$I = \frac{50}{L + 125} < 30\%$$

The impact factor was calculated to be 21.4% for Standard design example.

A.2.5.2.4
Shear Forces and Bending Moments
A.2.5.2.4.1
Due to Truck load

The maximum shear forces V and bending moments M due to HS 20 truck loading for all limit states except for fatigue limit state on a per-lane-basis are calculated using the following formulas given in the *PCI Design Manual* (PCI 2003).

Maximum bending moment due to HS 20 truck load

For $x/L = 0 – 0.333$

$$M = \frac{72(x)[(L - x) - 9.33]}{L}$$

For $x/L = 0.333 – 0.5$

$$M = \frac{72(x)[(L - x) - 4.67]}{L} - 112$$

Maximum shear force due to HS 20 truck load

For $x/L = 0 – 0.5$

$$V = \frac{72[(L - x) - 9.33]}{L}$$

where

x = Distance from the center of bearing to the section at which bending moment or shear force is calculated, ft.

L = Design span length = 108.583 ft.

Distributed bending moment due to truck load including dynamic load allowance (M_{LT}) is calculated as follows:

M_{LT} = (Moment per lane due to truck load)(DFM)(1+IM/100)
= (M)(0.639)(1 + 33/100)
= (M)(0.85)

Distributed shear force due to truck load including dynamic load allowance (V_{LT}) is calculated as follows:

V_{LT} = (Shear force per lane due to truck load)(DFV)(1+IM/100)
= (V)(0.814)(1 + 33/100)
= (V)(1.083)

where

M = Maximum bending moment due to HS 20 truck load, k-ft.

DFM = Distribution factor for live load per lane for moment in interior girders.

IM = Dynamic load allowance, applied to truck load or tandem load only.

DFV = Distribution factor for live load per lane for shear in interior girders.

V = Maximum shear force due to HS 20 truck load, kips.

The maximum bending moments and shear forces due to HS 20 truck load are calculated at every tenth of the span and at critical section for shear and hold down point section. The values are presented in Table A.2.5.2.

A.2.5.2.4.1 Due to Design Lane Load

The maximum bending moments (M_L) and shear forces (V_L) due to uniformly distributed lane load of 0.64 klf are calculated using the following formulas given by *PCI Design Manual* (PCI 2003).

Maximum bending moment, $M_L = 0.5(0.64)(x)(L - x)$

where

x = Distance from the center of bearing to the section at which bending moment or shear force is calculated, ft.

L = Design span length = 108.583 ft.

Maximum shear force, $V_L = \dfrac{0.32(L - x)^2}{L}$ for $x \leq 0.5L$

(Note that maximum shear force at a section is calculated at a section by placing the uniform load on the right of the section considered as given in *PCI Design Manual* (PCI 2003). This method yields a slightly conservative estimate of the shear force as compared to the shear force at a section under uniform load placed on the entire span length)

Distributed bending moment due to lane load (M_{LL}) is calculated as follows:

M_{LL} = (Moment per lane due to lane load)(DFM)
= M_L (0.639)

Distributed shear force due to lane load (V_{LL}) is calculated as follows:

V_{LL} = (shear force per lane due to lane load)(DFV)
= V_L (0.814)

where

M_L = Maximum bending moment due to lane load, k-ft.

DFM = Distribution factor for live load per lane for moment in interior girders.

DFV = Distribution factor for live load per lane for shear in interior girders.

V_L = Maximum shear force due to lane load, kips.

The maximum bending moments and shear forces due to lane load are calculated at every tenth of the span and at critical section for shear and hold down point section. The values are presented in Table A.2.5.2.

Table A.2.5.2. Shear Forces and Bending Moments due to Live Load

Distance x	Section x/L	HS 20 Truck Loading				Lane loading			
		Undistributed Truck Load		Distributed Truck + Dynamic Load		Undistributed Lane Load		Distributed Lane Load	
		Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment
		V	M	V_{LT}	M_{LT}	V_L	M_L	V_{LL}	M_{LL}
ft.		kips	k-ft.	kips	k-ft.	kips	k-ft.	kips	k-ft.
0.000	0.000	65.81	0.00	71.25	0.00	34.75	0.00	28.28	0.00
2.875	0.026	63.91	183.73	69.19	156.15	32.93	97.25	26.81	62.14
10.858	0.100	58.61	636.43	63.45	540.88	28.14	339.55	22.91	216.97
21.717	0.200	51.41	1116.54	55.66	948.91	22.24	603.67	18.10	385.75
32.575	0.300	44.21	1440.25	47.86	1224.03	17.03	792.31	13.86	506.28
43.433	0.400	37.01	1629.82	40.07	1385.14	12.51	905.49	10.18	578.61
48.862	0.450 (*HD*)	33.41	1671.64	36.17	1420.68	10.51	933.79	8.56	596.69
54.292	0.500	29.81	1674.37	32.27	1423.00	8.69	943.22	7.07	602.72

A.2.5.3 Load Combinations

LRFD Art. 3.4.1 specifies the load factors and load combinations. Total factored load effect is specified to be taken as:

$$Q = \sum \eta_i \, \gamma_i \, Q_i \qquad \text{[LRFD Eq. 3.4.1-1]}$$

where

Q = Factored force effects.

γ_i = Load factor, a statistically based multiplier applied to force effects specified by LRFD Table 3.4.1-1.

Q_i = Unfactored force effects.

η_i = Load modifier, a factor relating to ductility, redundancy and operational importance.
= $\eta_D \, \eta_R \, \eta_I \geq 0.95$, for loads for which a maximum value of γ_i is appropriate [LRFD Eq. 1.3.2.1-2]
= $\dfrac{1}{\eta_D \, \eta_R \, \eta_I} \leq 1.0$, for loads for which a minimum value of γ_i is appropriate [LRFD Eq. 1.3.2.1-3]

η_D = A factor relating to ductility
= 1.00 for all limit states except strength limit state.

For strength limit state:

η_D $\geq$ 1.05 for nonductile components and connections.
= 1.00 for conventional design and details complying with LRFD Specifications.
$\geq$ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by LRFD Specifications.

η_D = 1.00 is used in this example for strength and service limit states as this design is considered to be conventional and complying with LRFD Specifications.

η_R = A factor relating to redundancy
= 1.00 for all limit states except strength limit state.

For strength limit state:

η_R $\geq$ 1.05 for nonredundant members.
= 1.00 for conventional levels of redundancy.
$\geq$ 0.95 for exceptional levels of redundancy.

η_R = 1.00 is used in this example for strength and service limit states as this design is considered to provide conventional level of redundancy to the structure.

η_I = A factor relating to operational importance.
= 1.00 for all limit states except strength limit state.

For strength limit state:

η_I $\geq$ 1.05 for important bridges.
= 1.00 for typical bridges.
$\geq$ 0.95 for relatively less important bridges.

η_I = 1.00 is used in this example for strength and service limit states as this example illustrates the design of a typical bridge.

$\eta_i = \eta_D\,\eta_R\,\eta_I$ = 1 in present case [LRFD Art. 1.3.2]

The notations used in the following section are defined as follows:

DC = Dead load of structural components and non-structural attachments.

DW = Dead load of wearing surface and utilities.

LL = Vehicular live load.

IM = Vehicular dynamic load allowance.

This design example considers only the dead and vehicular live loads. The wind load and the extreme event loads including earthquake and vehicle collision loads are not included in the design which is typical to the design of bridges in Texas. Various limit states and load combinations provided by LRFD Art. 3.4.1 are investigated and the following limit states are found to be applicable in present case:

Service I: This limit state is used for normal operational use of bridge. This limit state provides the general load combination for service limit state stress checks and applies to all conditions except Service III limit state. For prestressed concrete components, this load combination is used to check for compressive stresses. The load combination is presented as follows:

$Q = 1.00\ (DC + DW) + 1.00(LL + IM)$ [LRFD Table 3.4.1-1]

Service III: This limit state is a special load combination for service limit state stress checks that applies only to tension in prestressed concrete structures to control cracks. The load combination for this limit state is presented as follows:

$Q = 1.00(DC + DW) + 0.80(LL + IM)$ [LRFD Table 3.4.1-1]

Strength I: This limit state is the general load combination for strength limit state design relating to the normal vehicular use of the bridge without wind. The load combination is presented as follows:

[LRFD Table 3.4.1-1 and 2]

$Q = \gamma_P(DC) + \gamma_P(DW) + 1.75(LL + IM)$

γ_P = Load factor for permanent loads provided in Table A.2.5.3.1.

Table A.2.5.3.1. Load Factors for Permanent Loads

Type of Load	Load Factor, γ_P	
	Maximum	Minimum
DC: Structural components and non-structural attachments	1.25	0.90
DW: Wearing surface and utilities	1.50	0.65

The maximum and minimum load combinations for strength limit state, Strength I are presented as follows:

Maximum $Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)$
Minimum $Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)$

For simple span bridges, the maximum load factors produce maximum effects. However, minimum load factors are used for component dead loads (*DC*), and wearing surface load (*DW*) when dead load and wearing surface stresses are opposite to those of live load. In the present example the maximum load factors are used to investigate the ultimate strength limit state.

A.2.6 ESTIMATION OF REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stress at the bottom fiber of the girder at midspan section. The load combination for Service III limit state is used to evaluate the bottom fiber stresses at the midspan section. The calculation for compressive stress in the top fiber of the girder at midspan section under service loads is also shown in the following section. The compressive stress is evaluated using the load combination for Service I limit state.

A.2.6.1 Service Load Stresses at Midspan

Tensile stress at bottom fiber of the girder at midspan due to applied dead and live loads using load combination Service III

$$f_b = \frac{M_{DCN}}{S_b} + \frac{M_{DCC} + M_{DW} + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

Compressive stress at top fiber of the girder at midspan due to applied dead and live loads using load combination Service I

$$f_t = \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}}$$

where:

f_b = Concrete stress at the bottom fiber of the girder, ksi

f_t = Concrete stress at the top fiber of the girder, ksi

M_{DCN} = Moment due to non-composite dead loads, k-ft.
$= M_g + M_S$

M_g = Moment due to girder self-weight = 1,209.98 k-ft.

M_S = Moment due to slab weight = 1,179.03 k-ft.

M_{DCN} = 1,209.98 + 1,179.03 = 2,389.01 k-ft.

M_{DCC} = Moment due to composite dead loads except wearing surface load, k-ft.
= M_{barr}

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DCC} = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.

M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8,902.67 in.^{3}

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder
= 16,876.83 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.^{3}

Substituting the bending moments and section modulus values, stresses at bottom fiber (f_b) and top fiber (f_t) of the girder at midspan section are:

$$f_b = \frac{(2389.01)(12\text{ in./ft.})}{10521.33} + \frac{[160.64 + 188.64 + 0.8(1423.00 + 602.72)](12\text{ in./ft.})}{16876.83}$$

= 2.725 + 1.400 = 4.125 ksi (As compared to 4.024 ksi for design using Standard Specifications)

$$f_t = \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{[160.64 + 188.64 + 1423.00 + 602.72](12\text{ in./ft.})}{54083.9}$$

= 3.220 + 0.527 = 3.747 ksi (As compared to 3.626 ksi for design using Standard Specifications)

The stresses in the top and bottom fibers of the girder at the hold down point, midspan, and top fiber of the slab are calculated in a similar way as shown above and the results are summarized in Table A.2.6.1.

Table A.2.6.1 Summary of Stresses due to Applied Loads

Load	Stresses in Girder				Stresses in Slab
	Stress at Hold Down (*HD*)		Stress at Midspan		Stress at Midspan
	Top Fiber (psi)	Bottom Fiber (psi)	Top Fiber (psi)	Bottom Fiber (psi)	Top Fiber (psi)
Girder self weight	1614.63	-1366.22	1630.94	-1380.03	-
Slab weight	1573.33	-1331.28	1589.22	-1344.73	-
Barrier weight	35.29	-113.08	35.64	-114.22	57.84
Wearing surface weight	41.44	-132.79	41.85	-134.13	67.93
Total dead load	3264.68	-2943.38	3297.66	-2973.10	125.77
HS 20 Truck load (multiplied by 0.8 for bottom fiber stress calculation)	315.22	-808.12	315.73	-809.44	512.40
Lane load (multiplied by 0.8 for bottom fiber stress calculation)	132.39	-339.41	133.73	-342.84	217.03
Total live load	447.61	-1147.54	449.46	-1152.28	729.43
Total load	3712.29	-4090.91	3747.12	-4125.39	855.21

(Negative values indicate tensile stress)

A.2.6.2 Allowable Stress Limit

LRFD Table 5.9.4.2.2-1 specifies the allowable tensile stress in fully prestressed concrete members. For members with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions (these corrosion conditions are assumed in this design), the allowable tensile stress at service limit state after losses is given as:

$$F_b = 0.19\sqrt{f'_c}$$

where

f'_c = Compressive strength of girder concrete at service = 5.0 ksi

F_b = $0.19\sqrt{5.0}$ = 0.4248 ksi (As compared to allowable tensile stress of 0.4242 ksi for the Standard design).

A.2.6.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – Allowable tensile stress at service = $f_b - F_b$

$f_{pb\text{-}reqd.} = 4.125 - 0.4248 = 3.700$ ksi

Assuming the eccentricity of the prestressing strands at midspan (e_c) as the distance from the centroid of the girder to the bottom fiber of the girder (PSTRS 14 methodology, TxDOT 2004)

$e_c = y_b = 24.75$ in.

Stress at bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}\, e_c}{S_b}$$

where:

P_{pe} = Effective prestressing force after all losses, kips

A = Area of girder cross section = 788.4 in.^{2}

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

Required prestressing force is calculated by substituting the corresponding values in above equation as follows.

$$3.700 = \frac{P_{pe}}{788.4} + \frac{24.75\, P_{pe}}{10521.33}$$

Solving for P_{pe},

$P_{pe} = 1021.89$ kips

Assuming final losses = 20% of initial prestress f_{pi} (TxDOT 2001)

Assumed final losses = 0.2(202.5) = 40.5 ksi

The prestressing force per strand after losses
= (cross sectional area of one strand) [f_{pi} – losses]
= 0.153(202.5 – 40.5) = 24.78 kips

Number of prestressing strands required = 1021.89/24.78 = 41.24

Try 42 – ½ in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6)+6(8)}{42} = 20.18 \text{ in.}$$

Available prestressing force

$P_{pe} = 42(24.78) = 1040.76$ kips

Stress at bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{1040.76}{788.4} + \frac{20.18(1040.76)}{10521.33}$$

$$= 1.320 + 1.996 = 3.316 \text{ ksi} < f_{pb\text{-}reqd.} = 3.700 \text{ ksi} \quad \text{(N.G.)}$$

Try 44 – ½ in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6)+8(8)}{44} = 20.02 \text{ in.}$$

Available prestressing force

$P_{pe} = 44(24.78) = 1090.32$ kips

Stress at bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{1090.32}{788.4} + \frac{20.02(1090.32)}{10521.33}$$

$$= 1.383 + 2.075 = 3.458 \text{ ksi} < f_{pb\text{-}reqd.} = 3.700 \text{ ksi} \quad \text{(N.G.)}$$

Try 46 – ½ in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6)+10(8)}{46} = 19.88 \text{ in.}$$

Available prestressing force

$P_{pe} = 46(24.78) = 1139.88$ kips

Stress at bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{1139.88}{788.4} + \frac{19.88(1139.88)}{10521.33}$$

$$= 1.446 + 2.154 = 3.600 \text{ ksi} < f_{pb\text{-}reqd.} = 3.700 \text{ ksi} \quad \text{(N.G.)}$$

Try 48 – ½ in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8)+2(10)}{48} = 19.67 \text{ in.}$$

Available prestressing force

$P_{pe} = 48(24.78) = 1189.44$ kips

Stress at bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{1189.44}{788.4} + \frac{19.67(1189.44)}{10521.33}$$

$$= 1.509 + 2.223 = 3.732 \text{ ksi} > f_{pb\text{-}reqd.} = 3.700 \text{ ksi} \quad \text{(O.K.)}$$

Therefore use 48 strands are used as a preliminary estimate for the number of strands. The strand arrangement is shown in Figure A.2.6.1.

Number of Strands	Distance from bottom fiber (in.)
2	10
10	8
12	6
12	4
12	2

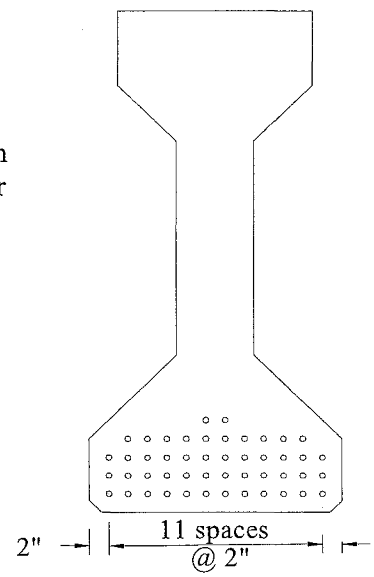

Fig. A.2.6.1 Initial Strand Arrangement

The distance from the center of gravity of the strands to the bottom fiber of the girder (y_{bs}) is calculated as:

$y_{bs} = y_b - e_c = 24.75 – 19.67 = 5.08$ in.

A.2.7 PRESTRESS LOSSES

[LRFD Art. 5.9.5]

The LRFD Specifications specifies formulas to determine the instantaneous losses. For time-dependent losses, two different options are provided. The first option is to use lump-sum estimate of time-dependent losses given by LRFD Art. 5.9.5.3. The second option is to use refined estimates for time-dependent losses given by LRFD Art. 5.9.5.4. The refined estimates are used in this design as they yield more accuracy as compared to lump-sum method.

The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pR1})$$

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

The TxDOT methodology was used for the evaluation of instantaneous prestress loss in Standard Design given by the following expression.

$$\Delta f_{pi} = (ES + \frac{1}{2} CR_S)$$

where:

Δf_{pi}	= Instantaneous prestress loss, ksi
Δf_{pES}	= Prestress loss due to elastic shortening, ksi
Δf_{pR1}	= Prestress loss due to steel relaxation before transfer, ksi
f_{pj}	= Jacking stress in prestressing strands = 202.5 ksi
ES	= Prestress loss due to elastic shortening, ksi
CR_S	= Prestress loss due to steel relaxation at service, ksi

The time-dependent loss of prestress is estimated using the following expression

$$\text{Time Dependent loss} = \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

where:

Δf_{pSR} = Prestress loss due to concrete shrinkage, ksi

Δf_{pCR} = Prestress loss due to concrete creep, ksi

Δf_{pR2} = Prestress loss due to steel relaxation after transfer, ksi

The total prestress loss in prestressed concrete members prestressed in a single stage, relative to stress immediately before transfer is given as:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$ [LRFD Eq. 5.9.5.1-1]

However considering the steel relaxation loss before transfer Δf_{pR1}, the total prestress loss is calculated using the following expression:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$$

The calculation of prestress loss due to elastic shortening, steel relaxation before and after transfer, creep of concrete and shrinkage of concrete are shown in following sections.

Trial number of strands = 48

A number of iterations based on TxDOT methodology (TxDOT 2001) will be performed to arrive at the optimum number of strands, required concrete strength at release (f'_{ci}) and required concrete strength at service (f'_c).

A.2.7.1 Iteration 1

A.2.7.1.1 Elastic Shortening

[LRFD Art. 5.9.5.2.3]

The loss in prestress due to elastic shortening in prestressed members is given as

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Eq. 5.9.5.2.3a-1]

where:

E_p = Modulus of elasticity of prestressing steel = 28500 ksi

E_{ci} = Modulus of elasticity of girder concrete at transfer, ksi
= $33000(w_c)^{1.5}\sqrt{f'_{ci}}$ [LRFD Eq. 5.4.2.4-1]

w_c = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.3.4-1 to be applicable)
= 0.150 kcf

f'_{ci} = Initial estimate of compressive strength of girder concrete at release = 4 ksi

E_{ci} = $[33000(0.150)^{1.5}\sqrt{4}\,]$ = 3834.25 ksi

f_{cgp} = Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self weight of the member at sections of maximum moment, ksi

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

P_i = Pretension force after allowing for the initial losses, kips

A = Area of girder cross-section = 788.4 in.^{2}

I = Moment of inertia of the non-composite section = 260403 in.^{4}

e_c = Eccentricity of the prestressing strands at the midspan = 19.67 in.

M_g = Moment due to girder self-weight at midspan, k-ft. = 1209.98 k-ft.

LRFD Art. 5.9.5.2.3a states that for pretension components of usual design, f_{cgp} can be calculated on the basis of prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, TxDOT methodology is to assume the initial losses as a percentage of the initial prestressing stress before release, f_{pi}. In both procedures initial losses assumed has to be checked, and if different from the assumed value a second iteration should be carried out.

The TxDOT methodology is used in this example and initial loss of 8% of initial prestress f_{pi} is assumed.

P_i = Pretension force after allowing for the 8% initial loss, kips
= (number of strands)(area of each strand)$[0.92(f_{pi})]$
= 48(0.153)(0.92)(202.5) = 1368.19 kips

$$f_{cgp} = \frac{1368.19}{788.4} + \frac{1368.19(19.67)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.67)}{260403}$$

$$= 1.735 + 2.033 - 1.097 = 2.671 \text{ ksi}$$

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{3834.25}\right](2.671) = 19.854 \text{ ksi}$$

A.2.7.1.2 Concrete Shrinkage

[LRFD Art. 5.9.5.4.2]

The loss is prestress due to concrete shrinkage for pretensioned members is given as:

$$\Delta f_{pSR} = 17 - 0.15\,H \qquad \text{[LRFD Eq. 5.9.5.4.2-1]}$$

where:

H = Average annual ambient relative humidity = 60%

Δf_{pSR} = [17 – 0.15(60)] = 8.0 ksi

A.2.7.1.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

The loss is prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0 \qquad \text{[LRFD Eq. 5.9.5.4.3-1]}$$

where:

Δf_{cdp} = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as f_{cgp}

$$= \frac{M_S\, e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_S = Moment due to slab weight at midspan section = 1179.03 k-ft.

M_{SDL} = Moment due to superimposed dead load = $M_{barr} + M_{DW}$

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

M_{SDL} = 160.64 + 188.64 = 349.28 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder
= 24.75 – 19.67 = 5.08 in.

I = Moment of inertia of the non-composite section
= 260403 in.^{4}

I_c = Moment of inertia of composite section = 694599.5 in.^{4}

$$\Delta f_{cdp} = \frac{1179.03(12\text{ in./ft.})(19.67)}{260403} + \frac{(349.28)(12\text{ in./ft.})(41.157 - 5.08)}{694599.5}$$

= 1.069 + 0.218 = 1.287 ksi

Prestress loss due to creep of concrete is
Δf_{pCR} = 12(2.671) – 7(1.287) = 23.05 ksi

A.2.7.1.4 Relaxation of Prestressing Strands

[LRFD Art. 5.9.5.4.4]

A.2.7.1.4.1 Relaxation at Transfer

[LRFD Art. 5.9.5.4.4b]

For pretensioned members, the relaxation loss is low-relaxation prestressing steel, initially stressed in excess of $0.5f_{pu}$ is given as:

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right]f_{pj}$$ [LRFD Eq. 5.9.5.4.4b-2]

where:

Δf_{pR1} = Prestress loss due to relaxation of steel before transfer, ksi

f_{pu} = Ultimate stress in prestressing steel = 270 ksi

f_{pj} = Initial stress in tendon at the end of stressing
= $0.75f_{pu}$ = 0.75(270) = 202.5 ksi > $0.5f_{pu}$ = 135 ksi

t = Time estimated in days from stressing to transfer taken as 1 day (default value for PSTRS14 design program (TxDOT 2004))

f_{py} = Yield strength of prestressing steel = 243 ksi

Prestress loss due to initial steel relaxation is

$$\Delta f_{pR1} = \frac{\log(24.0)(1)}{40}\left[\frac{202.5}{243} - 0.55\right]202.5 = 1.98\text{ ksi}$$

A.2.7.1.4.2 Relaxation After Transfer

[LRFD Art. 5.9.5.4.4c]

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

Δf_{pR2} = 30% of [20.0 – 0.4 Δf_{pES} – 0.2(Δf_{pSR} + Δf_{pCR})

[LRFD Art. 5.9.5.4.4c-1]

where the variables are same as defined in Section A.2.7 expressed in ksi units

$\Delta f_{pR2} = 0.3[20.0 - 0.4(19.854) - 0.2(8.0 + 23.05)] = 1.754$ ksi

The instantaneous loss of prestress is estimated using the following expression:

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$

$= 19.854 + 1.980 = 21.834$ ksi

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(19.854 + 1.980)}{202.5} = 10.78\% > 8\%$$ (assumed value of initial prestress loss)

Therefore another trial is required assuming 10.78% initial prestress loss.

The change in initial prestress loss will not affect the prestress losses due to concrete shrinkage (Δf_{pSR}) and initial steel relaxation (Δf_{pR1}). Therefore, the new trials will involve updating the losses due to elastic shortening (Δf_{pES}), creep of concrete (Δf_{pCR}), and steel relaxation after transfer (Δf_{pR2}).

Based on the initial prestress loss value of 10.78%, the pretension force after allowing for the initial losses is calculated as follows.

P_i = (number of strands)(area of each strand)[0.8922(f_{pi})]

= 48(0.153)(0.8922)(202.5) = 1326.84 kips

Loss in prestress due to elastic shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$= \frac{1326.84}{788.4} + \frac{1326.84(19.67)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.67)}{260403}$$

$$= 1.683 + 1.971 - 1.097 = 2.557 \text{ ksi}$$

$E_{ci} = 3834.25$ ksi

$E_p = 28500$ ksi

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{3834.25}\right](2.557) = 19.01 \text{ ksi}$$

The loss is prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0$$

The value of Δf_{cdp} depends on the dead load moments, superimposed dead load moments and section properties. Thus this value will not change with the change in initial prestress value and will be same as calculated in Section A.2.7.1.3.

$\Delta f_{cdp} = 1.287$ ksi

$$\Delta f_{pCR} = 12(2.557) - 7(1.287) = 21.675 \text{ ksi}$$

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:

Δf_{pR2} = 30% of [20.0 – 0.4 Δf_{pES} – 0.2(Δf_{pSR} + Δf_{pCR})

= 0.3[20.0 – 0.4(19.01) – 0.2(8.0 + 21.675)] = 1.938 ksi

The instantaneous loss of prestress is estimated using the following expression:

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$

= 19.01 + 1.980 = 20.99 ksi

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(19.01 + 1.980)}{202.5} = 10.37\% < 10.78\% \text{ (assumed value of initial prestress loss)}$$

Therefore another trial is required assuming 10.37% initial prestress loss.

Based on the initial prestress loss value of 10.37%, the pretension force after allowing for the initial losses is calculated as follows.

P_i = (number of strands)(area of each strand)[0.8963(f_{pi})]
= 48(0.153)(0.8963)(202.5) = 1332.94 kips

Loss in prestress due to elastic shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$= \frac{1332.94}{788.4} + \frac{1332.94(19.67)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.67)}{260403}$$

$$= 1.691 + 1.980 - 1.097 = 2.574 \text{ ksi}$$

E_{ci} = 3834.25 ksi

E_p = 28500 ksi

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{3834.25}\right](2.574) = 19.13 \text{ ksi}$$

The loss is prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0$$

Δf_{cdp} = 1.287 ksi

Δf_{pCR} = 12(2.574) – 7(1.287) = 21.879 ksi

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:

Δf_{pR2} = 30% of [20.0 – 0.4 Δf_{pES} – 0.2(Δf_{pSR} + Δf_{pCR})

= 0.3[20.0 – 0.4(19.13) – 0.2(8.0 + 21.879)] = 1.912 ksi

The instantaneous loss of prestress is estimated using the following expression:

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$

= 19.13 + 1.98 = 21.11 ksi

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(19.13 + 1.98)}{202.5} = 10.42\% \approx 10.37\%$$ (assumed value of initial prestress loss)

A.2.7.1.5 Total Losses at Transfer

Total prestress loss at transfer

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$

= 19.13 + 1.98 = 21.11 ksi

Effective initial prestress, f_{pi} = 202.5 – 21.11 = 181.39 ksi

P_i = Effective pretension after allowing for the initial prestress loss

= (number of strands)(area of each strand)(f_{pi})

= 48(0.153)(181.39) = 1332.13 kips

A.2.7.1.6 Total Losses at Service Loads

Total final loss in prestress

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pES} = Prestress loss due to elastic shortening = 19.13 ksi

Δf_{pSR} = Prestress loss due to concrete shrinkage = 8.0 ksi

Δf_{pCR} = Prestress loss due to concrete creep = 21.879 ksi

Δf_{pR1} = Prestress loss due to steel relaxation before transfer = 1.98 ksi

Δf_{pR2} = Prestress loss due to steel relaxation after transfer = 1.912 ksi

Δf_{pT} = 19.13 + 8.0 + 21.879 + 1.98 + 1.912 = 52.901 ksi

The percent final loss is calculated using the following expression:

$$\%\Delta f_{pT} = \frac{100(\Delta f_{pT})}{f_{pj}}$$

$$= \frac{100(52.901)}{202.5} = 26.12\%$$

Effective final prestress
$f_{pe} = f_{pi} - \Delta f_{pT}$ = 202.5 – 52.901 = 149.60 ksi

Check prestressing stress limit at service limit state (defined in Section A.2.3): $f_{pe} \leq 0.8 f_{py}$

f_{py} = Yield strength of prestressing steel = 243 ksi

f_{pe} = 149.60 ksi < 0.8(243) = 194.4 ksi (O.K.)

Effective prestressing force after allowing for final prestress loss

P_{pe} = (number of strands)(area of each strand)(f_{pe})

= 48(0.153)(149.60) = 1098.66 kips

A.2.7.1.7 Final Stresses at Midspan

The number of strands is updated based on the final stress at the bottom fiber of the girder at midspan section.

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress (f_{bf}) is calculated as follows:

$$f_{bf} = \frac{P_{pe}}{A} + \frac{P_{pe}\, e_c}{S_b}$$

$$= \frac{1098.66}{788.4} + \frac{1098.66(19.67)}{10521.33}$$

= 1.393 + 2.054 = 3.447 ksi < $f_{pb\text{-}reqd.}$ = 3.700 ksi (N.G)

($f_{pb\text{-}reqd.}$ calculations are presented in Section A.2.6.3)

Try 50 – ½ in. diameter, low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6)+10(8)+4(10)}{50} = 19.47 \text{ in.}$$

Effective pretension after allowing for the final prestress loss
$P_{pe} = 50(0.153)(149.60) = 1144.44$ kips

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress (f_{bf}) is:

$$f_{bf} = \frac{1144.44}{788.4} + \frac{1144.44(19.47)}{10521.33}$$

$= 1.452 + 2.118 = 3.57$ ksi $< f_{pb\text{-}reqd.} = 3.700$ ksi (N.G)

Try 52 – ½ in. diameter, low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6)+10(8)+6(10)}{52} = 19.29 \text{ in.}$$

Effective pretension after allowing for the final prestress loss
$P_{pe} = 52(0.153)(149.60) = 1190.22$ kips

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress (f_{bf}) is:

$$f_{bf} = \frac{1190.22}{788.4} + \frac{1190.22(19.29)}{10521.33}$$

$= 1.509 + 2.182 = 3.691$ ksi $< f_{pb\text{-}reqd.} = 3.700$ ksi (N.G)

Try 54 – ½ in. diameter, low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6)+10(8)+8(10)}{54} = 19.12 \text{ in.}$$

Effective pretension after allowing for the final prestress loss
$P_{pe} = 54(0.153)(149.60) = 1236.0$ kips

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress (f_{bf}) is:

$$f_{bf} = \frac{1236.0}{788.4} + \frac{1236.0(19.12)}{10521.33}$$

$= 1.567 + 2.246 = 3.813$ ksi $> f_{pb\text{-}reqd.} = 3.700$ ksi (O.K.)

Therefore use 54 – ½ in. diameter, 270 ksi low-relaxation strands.

Concrete stress at the top fiber of the girder due to effective prestress and applied permanent and transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + f_t = \frac{1236.0}{788.4} - \frac{1236.0(19.12)}{8902.67} + 3.747$$

$$= 1.567 - 2.654 + 3.747 = 2.66 \text{ ksi}$$

(f_t calculations are shown in Section A.2.6.1)

A.2.7.1.8 Initial Stresses at Hold Down Point

The concrete strength at release, f'_{ci}, is updated based on the initial stress at the bottom fiber of the girder at the hold down point.

Prestressing force after allowing for initial prestress loss

P_i = (number of strands)(area of strand)(effective initial prestress)

= 54(0.153)(181.39) = 1498.64 kips

(Effective initial prestress calculations are presented in Section A.2.7.1.5.)

Initial concrete stress at top fiber of the girder at the hold down point due to self weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i\,e_c}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at the hold down point based on overall girder length of 109'-8".
= $0.5wx(L - x)$

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Distance of hold down point from the end of the girder
= HD + (distance from centerline of bearing to the girder end)

HD = Hold down point distance from centerline of the bearing
= 48.862 ft. (see Sec. A.2.5.1.3)

x = 48.862 + 0.542 = 49.404 ft.

M_g = 0.5(0.821)(49.404)(109.67 - 49.404) = 1222.22 k-ft.

$$f_{ti} = \frac{1498.64}{788.4} - \frac{1498.64(19.12)}{8902.67} + \frac{1222.22(12\text{ in./ft.})}{8902.67}$$

$$= 1.901 - 3.218 + 1.647 = 0.330 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at the hold down point due to self-weight of the girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{1498.64}{788.4} + \frac{1498.64(19.12)}{10521.33} - \frac{1222.22(12\text{ in./ft.})}{10521.33}$$

$$= 1.901 + 2.723 - 1.394 = 3.230 \text{ ksi}$$

Compression stress limit for pretensioned members at transfer stage is $0.6 f'_{ci}$ [LRFD Art. 5.9.4.1.1]

Therefore, $f'_{ci\text{-reqd.}} = \dfrac{3230}{0.6} = 5383.33$ psi

A.2.7.2 Iteration 2

A second iteration is carried out to determine the prestress losses and subsequently estimate the required concrete strength at release and at service using the following parameters determined in the previous iteration.

Number of strands = 54
Concrete Strength at release, $f'_{ci} = 5383.33$ psi

A.2.7.2.1 Elastic Shortening

[LRFD Art. 5.9.5.2.3]

The loss in prestress due to elastic shortening in prestressed members is given as

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{[LRFD Eq. 5.9.5.2.3a-1]}$$

where:

E_p = Modulus of elasticity of prestressing steel = 28500 ksi

E_{ci} = Modulus of elasticity of girder concrete at transfer, ksi
$= 33000(w_c)^{1.5}\sqrt{f'_{ci}}$ [LRFD Eq. 5.4.2.4-1]

w_c = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.3.4-1 to be applicable)
= 0.150 kcf

f'_{ci} = Compressive strength of girder concrete at release
= 5.383 ksi

E_{ci} = $[33000(0.150)^{1.5}\sqrt{5.383}\,]$ = 4447.98 ksi

f_{cgp} = Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self weight of the member at sections of maximum moment, ksi

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

A = Area of girder cross-section = 788.4 in.^{2}

I = Moment of inertia of the non-composite section
= 260403 in.^{4}

e_c = Eccentricity of the prestressing strands at the midspan
= 19.12 in.

M_g = Moment due to girder self-weight at midspan, k-ft.
= 1209.98 k-ft.

P_i = Pretension force after allowing for the initial losses, kips

As the initial losses are dependent on the elastic shortening and the initial steel relaxation loss, which are yet to be determined, the initial loss value of 10.42% obtained in the last trial of iteration 1 is taken as an initial estimate for initial loss in prestress for this iteration.

P_i = (number of strands)(area of strand)[0.8958(f_{pi})]

= 54(0.153)(0.8958)(202.5) = 1498.72 kips

$$f_{cgp} = \frac{1498.72}{788.4} + \frac{1498.72(19.12)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.12)}{260403}$$

= 1.901 + 2.104 – 1.066 = 2.939 ksi

The prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{4447.98}\right](2.939) = 18.83\text{ ksi}$$

A.2.7.2.2 Concrete Shrinkage

[LRFD Art. 5.9.5.4.2]

The loss in prestress due to concrete shrinkage (Δf_{pSR}) depends on the relative humidity only. The change in compressive strength of girder concrete at release (f'_{ci}) and number of strands does not effect the prestress loss due to concrete shrinkage. It will remain the same as calculated in Section A.2.7.1.2.

Δf_{pSR} = 8.0 ksi

A.2.7.2.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

The loss is prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0 \qquad \text{[LRFD Eq. 5.9.5.4.3-1]}$$

where:

Δf_{cdp} = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as f_{cgp}.

$$= \frac{M_S\, e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_S = Moment due to slab weight at midspan section = 1179.03 k-ft.

M_{SDL} = Moment due to superimposed dead load = $M_{barr} + M_{DW}$

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

M_{SDL} = 160.64 + 188.64 = 349.28 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder = 24.75 – 19.12 = 5.63 in.

I = Moment of inertia of the non-composite section = 260403 in.^{4}

I_c = Moment of inertia of composite section = 694599.5 in.^{4}

$$\Delta f_{cdp} = \frac{1179.03(12\text{ in./ft.})(19.12)}{260403} + \frac{(349.28)(12\text{ in./ft.})(41.157 - 5.63)}{694599.5}$$

$$= 1.039 + 0.214 = 1.253 \text{ ksi}$$

Prestress loss due to creep of concrete is
$\Delta f_{pCR} = 12(2.939) - 7(1.253) = 26.50$ ksi

A.2.7.2.4 Relaxation of Prestressing Strands

[LRFD Art. 5.9.5.4.4]

A.2.7.2.4.1 Relaxation at Transfer

[LRFD Art. 5.9.5.4.4b]

The loss in prestress due to relaxation of steel at transfer (Δf_{pR1}) depends on the time from stressing to transfer of prestress (t), the initial stress in tendon at the end of stressing (f_{pj}) and the yield strength of prestressing steel (f_{py}). The change in compressive strength of girder concrete at release (f'_{ci}) and number of strands does not effect the prestress loss due to relaxation of steel before transfer. It will remain the same as calculated in Section A.2.7.1.4.1.

$\Delta f_{pR1} = 1.98$ ksi

A.2.7.2.4.2 Relaxation After Transfer

[LRFD Art. 5.9.5.4.4c]

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

$\Delta f_{pR2} = 30\%$ of $[20.0 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$
[LRFD Art. 5.9.5.4.4c-1]

where the variables are same as defined in Section A.2.7 expressed in ksi units

$\Delta f_{pR2} = 0.3[20.0 - 0.4(18.83) - 0.2(8.0 + 26.50)] = 1.670$ ksi

The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1} = 18.83 + 1.980 = 20.81 \text{ ksi}$$

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(18.83 + 1.98)}{202.5} = 10.28\% < 10.42\%$$ (assumed value of initial prestress loss)

Therefore another trial is required assuming 10.28% initial prestress loss.

The change in initial prestress loss will not affect the prestress losses due to concrete shrinkage (Δf_{pSR}) and initial steel relaxation (Δf_{pR1}). Therefore, the new trials will involve updating the losses due to elastic shortening (Δf_{pES}), creep of concrete (Δf_{pCR}), and steel relaxation after transfer (Δf_{pR2}).

Based on the initial prestress loss value of 10.28%, the pretension force after allowing for the initial losses is calculated as follows.

P_i = (number of strands)(area of each strand)[0.8972(f_{pi})]
= 54(0.153)(0.8972)(202.5) = 1501.06 kips

Loss in prestress due to elastic shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$= \frac{1501.06}{788.4} + \frac{1501.06(19.12)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.12)}{260403}$$

$$= 1.904 + 2.107 - 1.066 = 2.945 \text{ ksi}$$

E_{ci} = 4447.98 ksi

E_p = 28500 ksi

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{4447.98}\right](2.945) = 18.87 \text{ ksi}$$

The loss is prestress due to creep of concrete is given as:
$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0$

The value of Δf_{cdp} depends on the dead load moments, superimposed dead load moments and section properties. Thus this value will not change with the change in initial prestress value and will be same as calculated in Section A.2.7.2.3.
$\Delta f_{cdp} = 1.253$ ksi

$\Delta f_{pCR} = 12(2.945) - 7(1.253) = 26.57$ ksi

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:
Δf_{pR2} = 30% of $[20.0 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$

$= 0.3[20.0 - 0.4(18.87) - 0.2(8.0 + 26.57)] = 1.661$ ksi

The instantaneous loss of prestress is estimated using the following expression:
$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$
$= 18.87 + 1.98 = 20.85$ ksi

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(18.87 + 1.98)}{202.5} = 10.30\% \approx 10.28\%$$ (assumed value of initial prestress loss)

A.2.7.2.5 Total Losses at Transfer

Total prestress loss at transfer
$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$
$= 18.87 + 1.98 = 20.85$ ksi

Effective initial prestress, $f_{pi} = 202.5 - 20.85 = 181.65$ ksi

P_i = Effective pretension after allowing for the initial prestress loss

= (number of strands)(area of each strand)(f_{pi})

$= 54(0.153)(181.65) = 1500.79$ kips

A.2.7.2.6
Total Losses at Service Loads

Total final loss in prestress

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pES} = Prestress loss due to elastic shortening = 18.87 ksi

Δf_{pSR} = Prestress loss due to concrete shrinkage = 8.0 ksi

Δf_{pCR} = Prestress loss due to concrete creep = 26.57 ksi

Δf_{pR1} = Prestress loss due to steel relaxation before transfer = 1.98 ksi

Δf_{pR2} = Prestress loss due to steel relaxation after transfer = 1.661 ksi

Δf_{pT} = 18.87 + 8.0 + 26.57 + 1.98 + 1.661 = 57.08 ksi

The percent final loss is calculated using the following expression:

$$\%\Delta f_{pT} = \frac{100(\Delta f_{pT})}{f_{pj}}$$

$$= \frac{100(57.08)}{202.5} = 28.19\%$$

Effective final prestress

$f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 57.08 = 145.42$ ksi

Check prestressing stress limit at service limit state (defined in Section A.2.3): $f_{pe} \leq 0.8f_{py}$

f_{py} = Yield strength of prestressing steel = 243 ksi

f_{pe} = 145.42 ksi < 0.8(243) = 194.4 ksi (O.K.)

Effective prestressing force after allowing for final prestress loss

P_{pe} = (number of strands)(area of each strand)(f_{pe})

= 54(0.153)(145.42) = 1201.46 kips

A.2.7.2.7 Final Stresses at Midspan

The required concrete strength at service ($f'_{c\text{-reqd.}}$) is updated based on the final stresses at the top and bottom fibers of the girder at the midspan section shown as follows.

Concrete stresses at the top fiber of the girder at the midspan section due to transient loads, permanent loads and effective final prestress will be investigated for the following three cases using Service I limit state shown as follows.

1) Concrete stress at the top fiber of the girder at the midspan section due to effective final prestress + permanent loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}$$

where:

f_{tf} = Concrete stress at the top fiber of the girder, ksi

M_{DCN} = Moment due to non-composite dead loads, k-ft.
= $M_g + M_S$

M_g = Moment due to girder self-weight = 1,209.98 k-ft.

M_S = Moment due to slab weight = 1,179.03 k-ft.

M_{DCN} = 1,209.98 + 1,179.03 = 2,389.01 k-ft.

M_{DCC} = Moment due to composite dead loads except wearing surface load, k-ft.
= M_{barr}

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DCC} = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8,902.67 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.^{3}

$$f_{tf} = \frac{1201.46}{788.4} - \frac{1201.46(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64+188.64)(12\text{ in./ft.})}{54083.9}$$

$$= 1.524 - 2.580 + 3.220 + 0.077 = 2.241 \text{ ksi}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is 0.45 f_c'

$$f'_{c\text{ -reqd.}} = \frac{2241}{0.45} = 4980.0 \text{ psi} \quad \text{(controls)}$$

2) Concrete stress at the top fiber of the girder at the midspan section due to live load + ½ (effective final prestress + permanent loads)

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}\right)$$

where:

M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.

M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

$$f_{tf} = \frac{(1423+602.72)(12\text{ in./ft.})}{54083.9} + 0.5\left\{\frac{1201.46}{788.4} - \frac{1201.46(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64+188.64)(12\text{ in./ft.})}{54083.9}\right\}$$

$$= 0.449 + 0.5(1.524 - 2.580 + 3.220 + 0.077) = 1.570 \text{ ksi}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is 0.40 f_c'

$$f'_{c\text{ -reqd.}} = \frac{1570}{0.40} = 3925 \text{ psi}$$

3) Concrete stress at the top fiber of the girder at the midspan section due to effective prestress + permanent loads + transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}}$$

$$f_{tf} = \frac{1201.46}{788.4} - \frac{1201.46(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64)(12\text{ in./ft.})}{54083.9} + \frac{(1423 + 602.72)(12\text{ in./ft.})}{54083.9}$$

$$= 1.524 - 2.580 + 3.220 + 0.077 + 0.449 = 2.690 \text{ ksi}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is $0.60\,\phi_w f_c'$

where ϕ_w is the reduction factor, applicable to thin walled hollow rectangular compression members where the web or flange slenderness ratios are greater than 15.

[LRFD Art. 5.9.4.2.1]

The reduction factor ϕ_w is not defined for I-shaped girder cross sections and is taken as 1.0 in this design.

$$f'_{c\text{ -reqd.}} = \frac{2690}{0.60(1.0)} = 4483.33 \text{ psi}$$

Concrete stresses at the bottom fiber of the girder at the midspan section due to transient loads, permanent loads and effective final prestress will be investigated using Service III limit state as follows.

$f_{bf} = \dfrac{P_{pe}}{A} + \dfrac{P_{pe}\,e_c}{S_b} - f_b$ (f_b calculations are presented in Sec. A.2.6.1)

$$= \frac{1201.46}{788.4} + \frac{1201.46(19.12)}{10521.33} - 4.125$$

$$= 1.524 + 2.183 - 4.125 = -0.418 \text{ ksi}$$

Tensile stress limit in fully prestressed concrete members with bonded prestressing tendons, subjected to not worse than moderate corrosion conditions (assumed in this design example) at service limit state after losses is given by LRFD Table 5.9.4.2.2-1 as $0.19\sqrt{f_c'}$

$$f'_{c\text{ -reqd.}} = 1000\left(\frac{0.418}{0.19}\right)^2 = 4840.0 \text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations as shown above. The governing required concrete strength at service is 4980 psi.

A.2.7.2.8
Initial Stresses at Hold Down Point

Prestressing force after allowing for initial prestress loss

P_i = (number of strands)(area of strand)(effective initial prestress)

= 54(0.153)(181.65) = 1500.79 kips

(Effective initial prestress calculations are presented in Section A.2.7.2.5)

Initial concrete stress at top fiber of the girder at hold down point due to self weight of girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at hold down point based on overall girder length of 109'-8" = 1222.22 k-ft. (see Section A.2.7.1.8)

$$f_{ti} = \frac{1500.79}{788.4} - \frac{1500.79\,(19.12)}{8902.67} + \frac{1222.22\,(12\text{ in./ft.})}{8902.67}$$

= 1.904 – 3.223 + 1.647 = 0.328 ksi

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1500.79}{788.4} + \frac{1500.79\,(19.12)}{10521.33} - \frac{1222.22\,(12\text{ in./ft.})}{10521.33}$$

= 1.904 + 2.727 – 1.394 = 3.237 ksi

Compressive stress limit for pretensioned members at transfer stage is 0.60 f'_{ci} [LRFD Art.5.9.4.1.1]

$$f'_{ci\text{ -reqd.}} = \frac{3237}{0.60} = 5395 \text{ psi}$$

A.2.7.2.9 Initial Stresses at Girder End

The initial tensile stress at the top fiber and compressive stress at the bottom fiber of the girder at the girder end section are minimized by harping the web strands at the girder end. Following the TxDOT methodology (TxDOT 2001), the web strands are incrementally raised as a unit by two inches in each trial. The iterations are repeated until the top and bottom fiber stresses satisfies the allowable stress limits or the centroid of the topmost row of harped strands is at a distance of two inches from the top fiber of the girder in which case the concrete strength at release is updated based on the governing stress.

Table A.2.7.1 Summary of Top and Bottom Stresses at Girder End for Different Harped Strand Positions and Corresponding Required Concrete Strengths

Distance of the centroid of topmost row of harped web strands from		Eccentricity of prestressing strands at girder end (in.)	Top fiber stress (ksi)	Required concrete strength (ksi)	Bottom fiber stress (ksi)	Required concrete strength (ksi)
Bottom Fiber (in.)	Top Fiber (in.)					
10 (no harping)	44	19.12	-1.320	30.232	4.631	7.718
12	42	18.75	-1.257	27.439	4.578	7.630
14	40	18.38	-1.195	24.781	4.525	7.542
16	38	18.01	-1.132	22.259	4.472	7.454
18	36	17.64	-1.070	19.872	4.420	7.366
20	34	17.27	-1.007	17.620	4.367	7.278
22	32	16.90	-0.945	15.504	4.314	7.190
24	30	16.53	-0.883	13.523	4.261	7.102
26	28	16.16	-0.820	11.677	4.208	7.014
28	26	15.79	-0.758	9.967	4.155	6.926
30	24	15.42	-0.695	8.392	4.103	6.838
32	22	15.05	-0.633	6.952	4.050	6.750
34	20	14.68	-0.570	5.648	3.997	6.662
36	18	14.31	-0.508	4.479	3.944	6.574
38	16	13.93	-0.446	3.446	3.891	6.485
40	14	13.56	-0.383	2.548	3.838	6.397
42	12	13.19	-0.321	1.785	3.786	6.309
44	10	12.82	-0.258	1.157	3.733	6.221
46	8	12.45	-0.196	0.665	3.680	6.133
48	6	12.08	-0.133	0.309	3.627	6.045
50	4	11.71	-0.071	0.087	3.574	5.957
52	2	11.34	-0.008	0.001	3.521	5.869

The position of the harped web strands, eccentricity of strands at the girder end, top and bottom fiber stresses at the girder end, and the corresponding required concrete strengths are summarized in Table A.2.7.1. The required concrete strengths used in Table A.2.7.1 are based on the allowable stress limits at transfer stage specified in LRFD Art. 5.9.4.1 presented as follows.

Allowable compressive stress limit = $0.60\,f'_{ci}$

For fully prestressed members, in areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$ (f_y is the yield strength of nonprestressed reinforcement), not to exceed 30 ksi, the allowable tension at transfer stage is given as $0.24\sqrt{f'_{ci}}$

From Table A.2.7.1, it is evident that the web strands are needed to be harped to the top most position possible to control the bottom fiber stress at the girder end.

Detailed calculations for the case when 10 web strands (5 rows) are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of two inches from the top fiber of the girder) is presented as follows.

Eccentricity of prestressing strands at the girder end (see Fig. A.2.7.2)

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54}$$

$$= 11.34 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_e}{S_t}$$

$$= \frac{1500.79}{788.4} - \frac{1500.79\,(11.34)}{8902.67} = 1.904 - 1.912 = -0.008 \text{ ksi}$$

Tensile stress limit for fully prestressed concrete members with bonded reinforcement is $0.24\sqrt{f'_{ci}}$ [LRFD Art. 5.9.4.1]

$$f'_{ci\text{-reqd.}} = 1000\left(\frac{0.008}{0.24}\right)^2 = 1.11 \text{ psi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b}$$

$$= \frac{1500.79}{788.4} + \frac{1500.79\ (11.34)}{10521.33} = 1.904 + 1.618 = 3.522 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is $0.60\, f'_{ci}$ [LRFD Art. 5.9.4.1]

$$f'_{ci\text{ -reqd.}} = \frac{3522}{0.60} = 5870 \text{ psi} \qquad \text{(controls)}$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release, f'_{ci} = 5870 psi

Concrete strength at service, f'_c is greater of 4980 psi and f'_{ci}

f'_c = 5870 psi

A.2.7.3 Iteration 3

A third iteration is carried out to refine the prestress losses based on the updated concrete strengths. Based on the new prestress losses, the concrete strength at release and at service will be further refined.

Number of strands = 54

Concrete Strength at release, f'_{ci} = 5870 psi

A.2.7.3.1 Elastic Shortening

[LRFD Art. 5.9.5.2.3]

The loss in prestress due to elastic shortening in prestressed members is given as

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{[LRFD Eq. 5.9.5.2.3a-1]}$$

where:

E_p = Modulus of elasticity of prestressing steel = 28500 ksi

E_{ci} = Modulus of elasticity of girder concrete at transfer, ksi
= $33000(w_c)^{1.5}\sqrt{f'_{ci}}$ [LRFD Eq. 5.4.2.4-1]

w_c = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.3.4-1 to be applicable)
= 0.150 kcf

f'_{ci} = Compressive strength of girder concrete at release
= 5.870 ksi

E_{ci} = $[33000(0.150)^{1.5}\sqrt{5.870}\,]$ = 4644.83 ksi

f_{cgp} = Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self weight of the member at sections of maximum moment, ksi

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

A = Area of girder cross-section = 788.4 in.^{2}

I = Moment of inertia of the non-composite section
= 260403 in.^{4}

e_c = Eccentricity of the prestressing strands at the midspan
= 19.12 in.

M_g = Moment due to girder self-weight at midspan, k-ft.
= 1209.98 k-ft.

P_i = Pretension force after allowing for the initial losses, kips

As the initial losses are dependent on the elastic shortening and the initial steel relaxation loss, which are yet to be determined, the initial loss value of 10.30% obtained in the last trial (iteration 2) is taken as an initial estimate for initial loss in prestress for this iteration.

P_i = (number of strands)(area of strand)[0.897(f_{pi})]
= 54(0.153)(0.897)(202.5) = 1500.73 kips

$$f_{cgp} = \frac{1500.73}{788.4} + \frac{1500.73\ (19.12)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.12)}{260403}$$

= 1.904 + 2.107 – 1.066 = 2.945 ksi

The prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{4644.83}\right](2.945) = 18.07\text{ ksi}$$

A.2.7.3.2 Concrete Shrinkage

[LRFD Art. 5.9.5.4.2]

The loss in prestress due to concrete shrinkage (Δf_{pSR}) depends on the relative humidity only. The change in compressive strength of girder concrete at release (f'_{ci}) does not effect the prestress loss due to concrete shrinkage. It will remain the same as calculated in Section A.2.7.1.2.

Δf_{pSR} = 8.0 ksi

A.2.7.3.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

The loss is prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0 \qquad \text{[LRFD Eq. 5.9.5.4.3-1]}$$

where:

Δf_{cdp} = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as f_{cgp}.

$$= \frac{M_S\, e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_S = Moment due to slab weight at midspan section = 1179.03 k-ft.

M_{SDL} = Moment due to superimposed dead load = $M_{barr} + M_{DW}$

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

M_{SDL} = 160.64 + 188.64 = 349.28 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder = 24.75 – 19.12 = 5.63 in.

I = Moment of inertia of the non-composite section = 260403 in.^{4}

I_c = Moment of inertia of composite section = 694599.5 in.^{4}

$$\Delta f_{cdp} = \frac{1179.03(12\text{ in./ft.})(19.12)}{260403} + \frac{(349.28)(12\text{ in./ft.})(41.157 - 5.63)}{694599.5}$$

$$= 1.039 + 0.214 = 1.253 \text{ ksi}$$

Prestress loss due to creep of concrete is
$\Delta f_{pCR} = 12(2.945) - 7(1.253) = 26.57$ ksi

A.2.7.3.4 Relaxation of Prestressing Strands

[LRFD Art. 5.9.5.4.4]

A.2.7.3.4.1 Relaxation at Transfer

[LRFD Art. 5.9.5.4.4b]

The loss in prestress due to relaxation of steel at transfer (Δf_{pR1}) depends on the time from stressing to transfer of prestress (t), the initial stress in tendon at the end of stressing (f_{pj}) and the yield strength of prestressing steel (f_{py}). The change in compressive strength of girder concrete at release (f'_{ci}) and number of strands does not effect the prestress loss due to relaxation of steel before transfer. It will remain the same as calculated in Section A.2.7.1.4.1.

$\Delta f_{pR1} = 1.98$ ksi

A.2.7.3.4.2 Relaxation After Transfer

[LRFD Art. 5.9.5.4.4c]

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

$\Delta f_{pR2} = 30\%$ of $[20.0 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$

[LRFD Art. 5.9.5.4.4c-1]

where the variables are same as defined in Section A.2.7 expressed in ksi units

$\Delta f_{pR2} = 0.3[20.0 - 0.4(18.07) - 0.2(8.0 + 26.57)] = 1.757$ ksi

The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$$
$$= 18.07 + 1.980 = 20.05 \text{ ksi}$$

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(18.07 + 1.98)}{202.5} = 9.90\% < 10.30\% \text{ (assumed value of initial prestress loss)}$$

Therefore another trial is required assuming 9.90% initial prestress loss.

The change in initial prestress loss will not affect the prestress losses due to concrete shrinkage (Δf_{pSR}) and initial steel relaxation (Δf_{pR1}). Therefore, the new trials will involve updating the losses due to elastic shortening (Δf_{pES}), creep of concrete (Δf_{pCR}), and steel relaxation after transfer (Δf_{pR2}).

Based on the initial prestress loss value of 9.90%, the pretension force after allowing for the initial losses is calculated as follows.

P_i = (number of strands)(area of each strand)[0.901(f_{pi})]
= 54(0.153)(0.901)(202.5) = 1507.42 kips

Loss in prestress due to elastic shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$= \frac{1507.42}{788.4} + \frac{1507.42(19.12)^2}{260403} - \frac{1209.98(12\text{ in./ft.})(19.12)}{260403}$$

$$= 1.912 + 2.116 - 1.066 = 2.962 \text{ ksi}$$

E_{ci} = 4644.83 ksi

E_p = 28500 ksi

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[\frac{28500}{4644.83}\right](2.962) = 18.17 \text{ ksi}$$

The loss is prestress due to creep of concrete is given as:
$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0$

The value of Δf_{cdp} depends on the dead load moments, superimposed dead load moments and section properties. Thus this value will not change with the change in initial prestress value and will be same as calculated in Section A.2.7.2.3.
Δf_{cdp} = 1.253 ksi

Δf_{pCR} = 12(2.962) – 7(1.253) = 26.773 ksi

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:
Δf_{pR2} = 30% of [20.0 – 0.4 Δf_{pES} – 0.2(Δf_{pSR} + Δf_{pCR})

= 0.3[20.0 – 0.4(18.17) – 0.2(8.0 + 26.773)] = 1.733 ksi

The instantaneous loss of prestress is estimated using the following expression:
$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$
= 18.17 + 1.98 = 20.15 ksi

The percent instantaneous loss is calculated using the following expression:

$$\%\Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}}$$

$$= \frac{100(18.17 + 1.98)}{202.5} = 9.95\% \approx 9.90\%$$ (assumed value of initial prestress loss)

A.2.7.3.5 Total Losses at Transfer

Total prestress loss at transfer
$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$
= 18.17 + 1.98 = 20.15 ksi

Effective initial prestress, f_{pi} = 202.5 – 20.15 = 182.35 ksi

P_i = Effective pretension after allowing for the initial prestress loss

= (number of strands)(area of each strand)(f_{pi})

= 54(0.153)(182.35) = 1506.58 kips

A.2.7.3.6
Total Losses at Service Loads

Total final loss in prestress
$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pES} = Prestress loss due to elastic shortening = 18.17 ksi

Δf_{pSR} = Prestress loss due to concrete shrinkage = 8.0 ksi

Δf_{pCR} = Prestress loss due to concrete creep = 26.773 ksi

Δf_{pR1} = Prestress loss due to steel relaxation before transfer = 1.98 ksi

Δf_{pR2} = Prestress loss due to steel relaxation after transfer = 1.733 ksi

Δf_{pT} = 18.17 + 8.0 + 26.773 + 1.98 + 1.773 = 56.70 ksi

The percent final loss is calculated using the following expression:

$$\%\Delta f_{pT} = \frac{100(\Delta f_{pT})}{f_{pj}}$$

$$= \frac{100(56.70)}{202.5} = 28.0\%$$

Effective final prestress
$f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 56.70 = 145.80$ ksi

Check prestressing stress limit at service limit state (defined in Section A.2.3): $f_{pe} \leq 0.8 f_{py}$

f_{py} = Yield strength of prestressing steel = 243 ksi

f_{pe} = 145.80 ksi < 0.8(243) = 194.4 ksi (O.K.)

Effective prestressing force after allowing for final prestress loss

P_{pe} = (number of strands)(area of each strand)(f_{pe})
= 54(0.153)(145.80) = 1204.60 kips

A.2.7.3.7 Final Stresses at Midspan

The required concrete strength at service ($f'_{c\text{-reqd.}}$) is updated based on the final stresses at the top and bottom fibers of the girder at the midspan section shown as follows.

Concrete stresses at the top fiber of the girder at the midspan section due to transient loads, permanent loads and effective final prestress will be investigated for the following three cases using Service I limit state shown as follows.

1) Concrete stress at the top fiber of the girder at the midspan section due to effective final prestress + permanent loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}$$

where:

f_{tf} = Concrete stress at the top fiber of the girder, ksi

M_{DCN} = Moment due to non-composite dead loads, k-ft.
= $M_g + M_S$

M_g = Moment due to girder self-weight = 1,209.98 k-ft.

M_S = Moment due to slab weight = 1,179.03 k-ft.

M_{DCN} = 1,209.98 + 1,179.03 = 2,389.01 k-ft.

M_{DCC} = Moment due to composite dead loads except wearing surface load, k-ft.
= M_{barr}

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DCC} = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8,902.67 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.^{3}

$$f_{tf} = \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64+188.64)(12\text{ in./ft.})}{54083.9}$$

$$= 1.528 - 2.587 + 3.220 + 0.077 = 2.238 \text{ ksi}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is 0.45 f'_c

$$f'_{c\text{-reqd.}} = \frac{2238}{0.45} = 4973.33 \text{ psi} \quad \text{(controls)}$$

2) Concrete stress at the top fiber of the girder at the midspan section due to live load + ½ (effective final prestress + permanent loads)

$$f_{tf} = \frac{(M_{LT}+M_{LL})}{S_{tg}} + 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC}+M_{DW}}{S_{tg}}\right)$$

where:

M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.

M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

$$f_{tf} = \frac{(1423+602.72)(12\text{ in./ft.})}{54083.9} + 0.5\left\{\frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64+188.64)(12\text{ in./ft.})}{54083.9}\right\}$$

$$= 0.449 + 0.5(1.528 - 2.587 + 3.220 + 0.077) = 1.568 \text{ ksi}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is 0.40 f'_c

$$f'_{c\text{-reqd.}} = \frac{1568}{0.40} = 3920 \text{ psi}$$

3) Concrete stress at the top fiber of the girder at the midspan section due to effective prestress + permanent loads + transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC}+M_{DW}+M_{LT}+M_{LL}}{S_{tg}}$$

$$f_{tf} = \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67}$$
$$+ \frac{(160.64+188.64)(12\text{ in./ft.})}{54083.9} + \frac{(1423+602.72)(12\text{ in./ft.})}{54083.9}$$
$$= 1.528 - 2.587 + 3.220 + 0.077 + 0.449 = 2.687 \text{ ksi}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is $0.60\,\phi_w f'_c$

where ϕ_w is the reduction factor, applicable to thin walled hollow rectangular compression members where the web or flange slenderness ratios are greater than 15.

[LRFD Art. 5.9.4.2.1]

The reduction factor ϕ_w is not defined for I-shaped girder cross sections and is taken as 1.0 in this design.

$$f'_{c\text{ -reqd.}} = \frac{2687}{0.60(1.0)} = 4478.33 \text{ psi}$$

Concrete stresses at the bottom fiber of the girder at the midspan section due to transient loads, permanent loads and effective final prestress will be investigated using Service III limit state as follows.

$f_{bf} = \dfrac{P_{pe}}{A} + \dfrac{P_{pe}\,e_c}{S_b} - f_b$ (f_b calculations are presented in Sec. A.2.6.1)

$$= \frac{1204.60}{788.4} + \frac{1204.60(19.12)}{10521.33} - 4.125$$
$$= 1.528 + 2.189 - 4.125 = -0.408 \text{ ksi}$$

Tensile stress limit in fully prestressed concrete members with bonded prestressing tendons, subjected to not worse than moderate corrosion conditions (assumed in this design example) at service limit state after losses is given by LRFD Table 5.9.4.2.2-1 as $0.19\sqrt{f'_c}$

$$f'_{c\text{ -reqd.}} = 1000\left(\frac{0.408}{0.19}\right)^2 = 4611 \text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations as shown above. The governing required concrete strength at service is 4973.33 psi.

A.2.7.3.8 Initial Stresses at Hold Down Point

Prestressing force after allowing for initial prestress loss

P_i = (number of strands)(area of strand)(effective initial prestress)

= 54(0.153)(182.35) = 1506.58 kips

(Effective initial prestress calculations are presented in Section A.2.7.3.5)

Initial concrete stress at top fiber of the girder at hold down point due to self weight of girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at hold down point based on overall girder length of 109'-8" = 1222.22 k-ft. (see Section A.2.7.1.8)

$$f_{ti} = \frac{1506.58}{788.4} - \frac{1506.58\,(19.12)}{8902.67} + \frac{1222.22\,(12\text{ in./ft.})}{8902.67}$$

= 1.911 – 3.236 + 1.647 = 0.322 ksi

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1506.58}{788.4} + \frac{1506.58\,(19.12)}{10521.33} - \frac{1222.22\,(12\text{ in./ft.})}{10521.33}$$

= 1.911 + 2.738 – 1.394 = 3.255 ksi

Compressive stress limit for pretensioned members at transfer stage is 0.60 f'_{ci} [LRFD Art.5.9.4.1.1]

$$f'_{ci\text{-}reqd.} = \frac{3255}{0.60} = 5425 \text{ psi}$$

A.2.7.3.9 Initial Stresses at Girder End

The eccentricity of the prestressing strands at the girder end when 10 web strands are harped to the top most location (centroid of the top most row of harped strands is at a distance of two inches from the top fiber of the girder) is calculated as follows (see Fig. A.2.7.2).

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54}$$

$$= 11.34 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_e}{S_t}$$

$$= \frac{1506.58}{788.4} - \frac{1506.58\,(11.34)}{8902.67} = 1.911 - 1.919 = -0.008 \text{ ksi}$$

Tensile stress limit for fully prestressed concrete members with bonded reinforcement is $0.24\sqrt{f'_{ci}}$ [LRFD Art. 5.9.4.1]

$$f'_{ci\text{-reqd.}} = 1000\left(\frac{0.008}{0.24}\right)^2 = 1.11 \text{ psi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58\,(11.34)}{10521.33} = 1.911 + 1.624 = 3.535 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is $0.60\,f'_{ci}$ [LRFD Art. 5.9.4.1]

$$f'_{ci\text{-reqd.}} = \frac{3535}{0.60} = 5892 \text{ psi} \quad \text{(controls)}$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release, $f'_{ci} = 5892$ psi

Concrete strength at service, f'_c is greater of 4973 psi and f'_{ci}

$f'_c = 5892$ psi

The difference in the required concrete strengths at release and at service obtained from iterations 2 and 3 is almost 20 psi. Hence the concrete strengths are sufficiently converged and another iteration is not required.

Therefore provide:

f'_{ci} = 5892 psi (as compared to 5455 psi obtained for Standard design example, an increase of 8%)

f'_c = 5892 psi (as compared to 5583 psi obtained for Standard design example, an increase of 5.5%)

54 – ½ in. diameter, 10 draped at the end, GR 270 low-relaxation strands (as compared to 50 strands obtained for Standard design example, an increase of 8%)

The final strand patterns at the midspan section and at the girder ends are shown in Figures A.2.7.1 and A.2.7.2. The longitudinal strand profile is shown in Figure A.2.7.3.

No. of Strands	Distance from Bottom Fiber (in.)
8	10
10	8
12	6
12	4
12	2

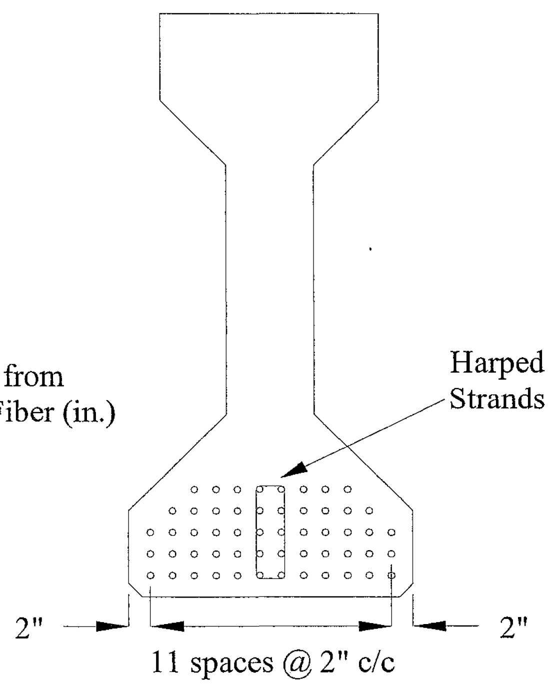

Fig.A.2.7.1 Final Strand Pattern at Midspan Section

No. of Strands	Distance from Bottom Fiber (in.)
2	52
2	50
2	48
2	46
2	44

No. of Strands	Distance from Bottom Fiber (in.)
6	10
8	8
10	6
10	4
10	2

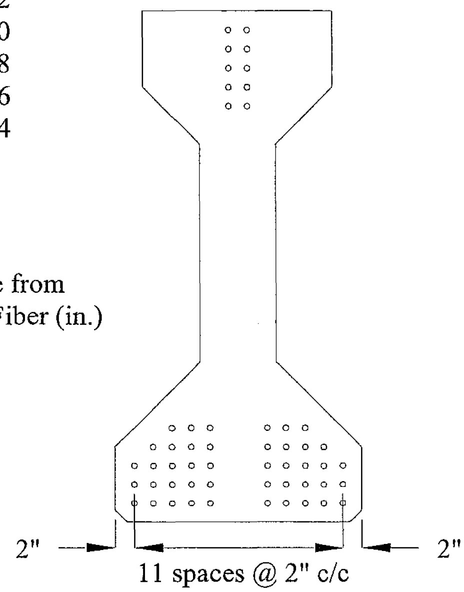

Fig. A.2.7.2 Final Strand Pattern at Girder End

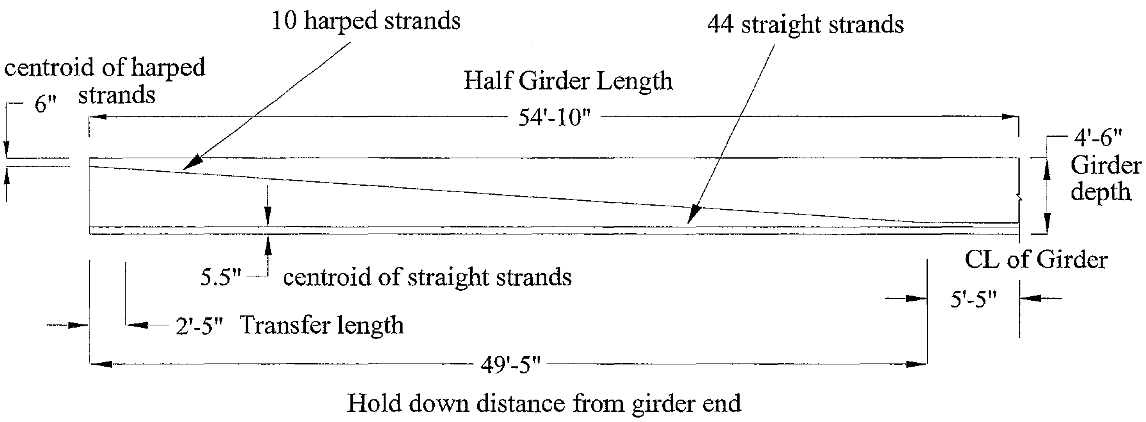

Fig. A.2.7.3 Longitudinal Strand Profile (half of the girder length is shown)

The distance between the centroid of the 10 harped strands and the top fiber of the girder the girder end

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

The distance between the centroid of the 10 harped strands and the bottom fiber of the girder at the harp points

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

Transfer length distance from girder end = 60(strand diameter)

[LRFD Art. 5.8.2.3]

Transfer length = 60(0.50) = 30 in. = 2'-6"

The distance between the centroid of 10 harped strands and the top of the girder at the transfer length section

$$= 6 \text{ in.} + \frac{(54 \text{ in.} - 6 \text{ in.} - 6 \text{ in.})}{49.4 \text{ ft.}}(2.5 \text{ ft.}) = 8.13 \text{ in.}$$

The distance between the centroid of the 44 straight strands and the bottom fiber of the girder at all locations

$$= \frac{10(2) + 10(4) + 10(6) + 8(8) + 6(10)}{44} = 5.55 \text{ in.}$$

A.2.8 STRESS SUMMARY

A.2.8.1 Concrete Stresses at Transfer

A.2.8.1.1 Allowable Stress Limits

[LRFD Art. 5.9.4]

The allowable stress limits at transfer for fully prestressed components, specified by the LRFD Specifications are as follows

Compression: $0.6 f'_{ci} = 0.6(5892) = +3535$ psi = 3.535 ksi (comp.)

Tension: The maximum allowable tensile stress for fully prestressed components is specified as follows:

- In areas other than the precompressed tensile zone and without bonded reinforcement: $0.0948\sqrt{f'_{ci}} \leq 0.2$ ksi.

  $0.0948\sqrt{f'_{ci}} = 0.0948\sqrt{5.892} = 0.23 \text{ ksi} > 0.2 \text{ ksi}$

  Allowable tension without bonded reinforcement = – 0.2 ksi

- In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$, not to exceed 30 ksi (see LRFD C 5.9.4.1.2):

$$0.24\sqrt{f'_{ci}} = 0.24\sqrt{5.892} = -\ 0.582 \text{ ksi (tension)}$$

A.2.8.1.2 Stresses at Girder Ends

Stresses at the girder ends are checked only at transfer, because it almost always governs.

The eccentricity of the prestressing strands at the girder end when 10 web strands are harped to the top most location (centroid of the top most row of harped strands is at a distance of two inches from the top fiber of the girder) is calculated as follows (see Fig. A.2.7.2).

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54}$$

$$= 11.34 \text{ in.}$$

Prestressing force after allowing for initial prestress loss

P_i = (number of strands)(area of strand)(effective initial prestress)

= 54(0.153)(182.35) = 1506.58 kips

(Effective initial prestress calculations are presented in Section A.2.7.3.5)

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_e}{S_t}$$

$$= \frac{1506.58}{788.4} - \frac{1506.58\,(11.34)}{8902.67} = 1.911 - 1.919 = -\ 0.008 \text{ ksi}$$

Allowable tension without additional bonded reinforcement is $-\ 0.20$ ksi $< -\ 0.008$ ksi (reqd.) (O.K.)

(The additional bonded reinforcement is not required in this case, but where necessary, required area of reinforcement can be calculated using LRFD C 5.9.4.1.2.)

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58\,(11.34)}{10521.33} = 1.911 + 1.624 = +3.535 \text{ ksi}$$

Allowable compression: +3.535 ksi = +3.535 ksi (reqd.) (O.K.)

A.2.8.1.3 Stresses at Transfer Length Section

Stresses at transfer length are checked only at release, because it almost always governs.

Transfer length = 60(strand diameter) [LRFD Art. 5.8.2.3]
= 60(0.5) = 30 in. = 2'-6"

The transfer length section is located at a distance of 2'-6" from the end of the girder or at a point 1'-11.5" from the centerline of the bearing support as the girder extends 6.5 in. beyond the bearing centerline. Overall girder length of 109'-8" is considered for the calculation of bending moment at the transfer length section.

Moment due to girder self-weight, $M_g = 0.5wx(L - x)$

where:

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Transfer length distance from girder end = 2.5 ft.

$M_g = 0.5(0.821)(2.5)(109.67 - 2.5) = 109.98$ k–ft.

Eccentricity of prestressing strands at transfer length section

$$e_t = e_c - (e_c - e_e)\,\frac{(49.404 - x)}{49.404}$$

where:

e_c = Eccentricity of prestressing strands at midspan = 19.12 in.

e_e = Eccentricity of prestressing strands at girder end = 11.34 in.

x = Distance of transfer length section from girder end = 2.5 ft.

$$e_t = 19.12 - (19.12 - 11.34)\frac{(49.404 - 2.5)}{49.404} = 11.73 \text{ in.}$$

Initial concrete stress at top fiber of the girder at the transfer length section due to self-weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_t}{S_t} + \frac{M_g}{S_t}$$

$$= \frac{1506.58}{788.4} - \frac{1506.58\,(11.73)}{8902.67} + \frac{109.98\,(12\text{ in./ft.})}{8902.67}$$

$$= 1.911 - 1.985 + 0.148 = +0.074 \text{ ksi}$$

Allowable compression: +3.535 ksi >> 0.074 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_t}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58\,(11.73)}{10521.33} - \frac{109.98\,(12\text{ in./ft.})}{10521.33}$$

$$= 1.911 + 1.680 - 0.125 = 3.466 \text{ ksi}$$

Allowable compression: +3.535 ksi > 3.466 ksi (reqd.) (O.K.)

A.2.8.1.4 Stresses at Hold Down Points

The eccentricity of the prestressing strands at the harp points is the same as at midspan.

$e_{harp} = e_c = 19.12$ in.

Initial concrete stress at top fiber of the girder at hold down point due to self-weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_{harp}}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at hold down point based on overall girder length of 109'-8" = 1222.22 k-ft. (see Section A.2.7.1.8)

$$f_{ti} = \frac{1506.58}{788.4} - \frac{1506.58\,(19.12)}{8902.67} + \frac{1222.22\,(12\text{ in./ft.})}{8902.67}$$

$$= 1.911 - 3.236 + 1.647 = 0.322 \text{ ksi}$$

Allowable compression: +3.535 ksi >> 0.322 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of the girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i\, e_{harp}}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58\,(19.12)}{10521.33} - \frac{1222.22\,(12\text{ in./ft.})}{10521.33}$$

$= 1.911 + 2.738 - 1.394 = 3.255$ ksi

Allowable compression: +3.535 ksi > 3.255 ksi (reqd.) (O.K.)

A.2.8.1.5 Stresses at Midspan

Bending moment due to girder self-weight at midspan section based on overall girder length of 109'-8"

$$M_g = 0.5wx(L - x)$$

where:

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Half the girder length = 54.84 ft.

$M_g = 0.5(0.821)(54.84)(109.67 - 54.84) = 1234.32$ k–ft.

Initial concrete stress at top fiber of the girder at midspan section due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i\, e_c}{S_t} + \frac{M_g}{S_t}$$

$$= \frac{1506.58}{788.4} - \frac{1506.58\,(19.12)}{8902.67} + \frac{1234.32\,(12\text{ in./ft.})}{8902.67}$$

$= 1.911 - 3.236 + 1.664 = 0.339$ ksi

Allowable compression: +3.535 ksi >> 0.339 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at midspan section due to self-weight of the girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i\, e_c}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58\,(19.12)}{10521.33} - \frac{1234.32\,(12\text{ in./ft.})}{10521.33}$$

$= 1.911 + 2.738 - 1.408 = 3.241$ ksi

Allowable compression: +3.535 ksi > 3.241 ksi (reqd.) (O.K.)

A.2.8.1.6 Stress Summary at Transfer

Allowable Stress Limits:

Compression: + 3.535 ksi

Tension: – 0.20 ksi without additional bonded reinforcement

– 0.582 ksi with additional bonded reinforcement

Stresses due to effective initial prestress and self-weight of the girder:

Location	Top of girder f_t (ksi)	Bottom of girder f_b (ksi)
Girder end	–0.008	+3.535
Transfer length section	+0.074	+3.466
Hold down points	+0.322	+3.255
Midspan	+0.339	+3.241

A.2.8.2 Concrete Stresses at Service Loads

A.2.8.2.1 Allowable Stress Limits

[LRFD Art. 5.9.4.2]

The allowable stress limits at service load after losses have occurred specified by the LRFD Specifications are presented as follows.

Compression:

Case (I): For stresses due to sum of effective prestress and permanent loads

$$0.45\,f_c' = 0.45(5892)/1000 = +2.651 \text{ ksi (for precast girder)}$$

$$0.45\,f_c' = 0.45(4000)/1000 = +1.800 \text{ ksi (for slab)}$$

(Note that the allowable stress limit for this case is specified as $0.40\,f_c'$ in Standard Specifications)

Case (II): For stresses due to live load and 0ne-half the sum of effective prestress and permanent loads

$$0.40\,f_c' = 0.40(5892)/1000 = +2.356 \text{ ksi (for precast girder)}$$

$$0.40\,f_c' = 0.40(4000)/1000 = +1.600 \text{ ksi (for slab)}$$

Case (III): For stresses due to sum of effective prestress, permanent loads and transient loads

$0.60\,f_c' = 0.60(5892)/1000 = +3.535$ ksi (for precast girder)

$0.60\,f_c' = 0.60(4000)/1000 = +2.400$ ksi (for slab)

Tension: For components with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions, for stresses due to load combination Service III

$$0.19\sqrt{f_c'} = 0.19\sqrt{5.892} = -\,0.461 \text{ ksi}$$

A.2.8.2.2 Final Stresses at Midspan

Effective prestressing force after allowing for final prestress loss

P_{pe} = (number of strands)(area of each strand)(f_{pe})

= 54(0.153)(145.80) = 1204.60 kips

(Calculations for effective final prestress (f_{pe}) are shown in Section A.2.7.3.6)

Concrete stresses at the top fiber of the girder at the midspan section due to transient loads, permanent loads and effective final prestress will be investigated for the following three cases using Service I limit state shown as follows.

Case (I): Concrete stress at the top fiber of the girder at the midspan section due to the sum of effective final prestress and permanent loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}$$

where:

f_{tf} = Concrete stress at the top fiber of the girder, ksi

M_{DCN} = Moment due to non-composite dead loads, k-ft.
$= M_g + M_S$

M_g = Moment due to girder self-weight = 1,209.98 k-ft.

M_S = Moment due to slab weight = 1,179.03 k-ft.

M_{DCN} = 1,209.98 + 1,179.03 = 2,389.01 k-ft.

M_{DCC} = Moment due to composite dead loads except wearing surface load, k-ft.
= M_{barr}

M_{barr} = Moment due to barrier weight = 160.64 k-ft.

M_{DCC} = 160.64 k-ft.

M_{DW} = Moment due to wearing surface load = 188.64 k-ft.

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8,902.67 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.^{3}

$$f_{tf} = \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64)(12\text{ in./ft.})}{54083.9}$$

$$= 1.528 - 2.587 + 3.220 + 0.077 = 2.238 \text{ ksi}$$

Allowable compression: +2.651 ksi > 2.238 ksi (reqd.) (O.K.)

Case (II): Concrete stress at the top fiber of the girder at the midspan section due to the live load and one-half the sum of effective final prestress and permanent loads

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}\right)$$

where:

M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.

M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

$$f_{tf} = \frac{(1423 + 602.72)(12\text{ in./ft.})}{54083.9} + 0.5\left\{\frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64)(12\text{ in./ft.})}{54083.9}\right\}$$

$$= 0.449 + 0.5(1.528 - 2.587 + 3.220 + 0.077) = 1.568 \text{ ksi}$$

Allowable compression: +2.356 ksi > 1.568 ksi (reqd.) (O.K.)

Case (III): Concrete stress at the top fiber of the girder at the midspan section due to sum of effective final prestress, permanent loads and transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe}\,e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}}$$

$$= \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12\text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64 + 1423.0 + 602.72)(12\text{ in./ft.})}{54083.9}$$

$$= 1.528 - 2.587 + 3.220 + 0.527 = 2.688 \text{ ksi}$$

Allowable compression: +3.535 ksi > 2.688 ksi (reqd.) (O.K.)

Concrete stresses at the bottom fiber of the girder at the midspan section due to transient loads, permanent loads and effective final prestress is investigated using Service III limit state as follows.

$$f_{bf} = \frac{P_{pe}}{A} + \frac{P_{pe}\,e_c}{S_b} - \frac{M_{DCN}}{S_b} - \frac{M_{DCC} + M_{DW} + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

where:

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder = 16,876.83 in.^{3}

$$f_{bf} = \frac{1204.60}{788.4} + \frac{1204.60(19.12)}{10521.33} - \frac{(2389.01)(12\text{ in./ft.})}{10521.33} - \frac{[160.64 + 188.64 + 0.8(1423.0 + 602.72)](12\text{ in./ft.})}{16876.83}$$

$$= 1.528 + 2.189 - 2.725 - 1.401 = -0.409 \text{ ksi}$$

Allowable tension: – 0.461 ksi < – 0.409 ksi (reqd.) (O.K.)

Superimposed dead loads and live loads contribute to the stresses at the top of the slab calculated as follows.

Case (I): Superimposed dead load effect

Concrete stress at the top fiber of the slab at midspan section due to superimposed dead loads

$$f_t = \frac{M_{DCC} + M_{DW}}{S_{tc}}$$

$$= \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{33325.31} = 0.126 \text{ ksi}$$

Allowable compression: +1.800 ksi >> +0.126 ksi (reqd.) (O.K.)

Case (II): Live load + 0.5(superimposed dead loads)

Concrete stress at the top fiber of the slab at midspan section due to sum of live loads and one-half the superimposed dead loads

$$f_t = \frac{M_{LT} + M_{LL} + 0.5(M_{DCC} + M_{DW})}{S_{tc}}$$

$$= \frac{[1423.0 + 602.72 + 0.5(160.64 + 188.64)](12 \text{ in./ft.})}{33325.31}$$

$$= +0.792 \text{ ksi}$$

Allowable compression: +1.600 ksi > +0.792 ksi (reqd.) (O.K.)

Case (III): Superimposed dead loads + Live load

Concrete stress at the top fiber of the slab at midspan section due to sum of permanent loads and live load.

$$f_t = \frac{M_{LT} + M_{LL} + M_{DCC} + M_{DW}}{S_{tc}}$$

$$= \frac{[1423.0 + 602.72 + 160.64 + 188.64](12 \text{ in./ft.})}{33325.31} = +0.855 \text{ ksi}$$

Allowable compression: +2.400 ksi > +0.855 ksi (reqd.) (O.K.)

A.2.8.2.3 Summary of Stresses at Service Loads

The final stresses at the top and bottom fiber of the girder and at the top fiber of the slab at service conditions for the cases defined in Section A.2.8.2.2 are summarized as follows.

At Midspan	Top of slab f_t (ksi)	Top of Girder f_t (ksi)	Bottom of girder f_b (ksi)
Case I	+0.126	+2.238	–
Case II	+0.792	+1.568	–
Case III	+0.855	+2.688	– 0 .409

A.2.8.2.4 Composite Section Properties

The composite section properties calculated in Section A.2.4.2.3 were based on the modular ratio value of 1. But as the actual concrete strength is now selected, the actual modular ratio can be determined and the corresponding composite section properties can be evaluated.

Modular ratio between slab and girder concrete

$$n = \left(\frac{E_{cs}}{E_{cp}}\right)$$

where:

n = Modular ratio between slab and girder concrete

E_{cs} = Modulus of elasticity of slab concrete, ksi
$= 33{,}000(w_c)^{1.5}\sqrt{f'_{cs}}$ [LRFD Eq. 5.4.2.4-1]

w_c = Unit weight of concrete = (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable)
= 0.150 kcf

f'_{cs} = Compressive strength of slab concrete at service
= 4.0 ksi

E_{cs} = $[33{,}000(0.150)^{1.5}\sqrt{4}\,] = 3{,}834.25$ ksi

E_{cp} = Modulus of elasticity of girder concrete at service, ksi
$= 33{,}000(w_c)^{1.5}\sqrt{f'_c}$

f'_c = Compressive strength of precast girder concrete at service
= 5.892 ksi

E_{cp} = [33,000(0.150)$^{1.5}\sqrt{5.892}$] = 4,653.53 ksi

$$n = \frac{3,834.25}{4,653.53} = 0.824$$

Transformed flange width, b_{tf} = n*(effective flange width)

Effective flange width = 96 in. (see Section A.2.4.2)

b_{tf} = 0.824*(96) = 79.10 in.

Transformed Flange Area, A_{tf} = n*(effective flange width)(t_s)

t_s = Slab thickness = 8 in.

A_{tf} = 0.824*(96)(8) = 632.83 in.^{2}

Table A.1.8.1. Properties of Composite Section

	Transformed Area A (in.^{2})	y_b in.	Ay_b in.^{3}	$A(y_{bc} - y_b)^2$	I in.^{4}	$I + A(y_{bc} - y_b)^2$ In.^{4}
Girder	788.40	24.75	19,512.9	172,924.58	260,403.0	433,327.6
Slab	632.83	58.00	36,704.1	215,183.46	3,374.9	218,558.4
Σ	1,421.23		56,217.0			651,886.0

A_c = Total area of composite section = 1,421.23 in.^{2}

h_c = Total height of composite section = 54 in. + 8 in. = 62 in.

I_c = Moment of inertia of composite section = 651,886.0 in^4

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.
= 56,217.0/1421.23 = 39.56 in.

y_{tg} = Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.
= 54 - 39.56 = 14.44 in.

y_{tc} = Distance from the centroid of the composite section to extreme top fiber of the slab = 62 - 39.56 = 22.44 in.

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.^{3}
= I_c/y_{bc} = 651,886.0/39.56 = 16,478.41 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder, in.^{3}
= I_c/y_{tg} = 651,886.0/14.44 = 45,144.46 in.^{3}

S_{tc} = Section modulus of composite section referenced to the top fiber of the slab, in.^{3}
= I_c/y_{tc} = 651,886.0/22.44 = 29,050.18 in.^{3}

A.2.9 CHECK FOR LIVE LOAD MOMENT DISTRIBUTION FACTOR

The live load moment distribution factor calculation involves a parameter for longitudinal stiffness, K_g. This parameter depends on the modular ratio between the girder and the slab concrete. The live load moment distribution factor calculated in Section A.2.5.2.2.1 is based on the assumption that the modular ratio between the girder and slab concrete is 1. However, as the actual concrete strength is now chosen, the live load moment distribution factor based on the actual modular ratio needs to be calculated and compared to the distribution factor calculated in Section A.2.5.2.2.1. If the difference between the two is found to be large, the bending moments have to be updated based on the calculated live load moment distribution factor.

$K_g = n(I + A\,e_g^2)$ [LRFD Art. 3.6.1.1.1]

where:

n = Modular ratio between girder and slab concrete.

$$= \frac{E_c \text{ for girder concrete}}{E_c \text{ for slab concrete}} = \left(\frac{E_{cp}}{E_{cs}}\right)$$

(Note that this ratio is the inverse of the one defined for composite section properties in Section A.2.8.2.4)

E_{cs} = Modulus of elasticity of slab concrete, ksi

$= 33{,}000(w_c)^{1.5}\sqrt{f'_{cs}}$ [LRFD Eq. 5.4.2.4-1]

w_c = Unit weight of concrete = (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable)
= 0.150 kcf

f'_{cs} = Compressive strength of slab concrete at service
= 4.0 ksi

$E_{cs} = [33{,}000(0.150)^{1.5}\sqrt{4}\,] = 3{,}834.25$ ksi

E_{cp} = Modulus of elasticity of girder concrete at service, ksi

$= 33{,}000(w_c)^{1.5}\sqrt{f'_c}$

f'_c = Compressive strength of precast girder concrete at service = 5.892 ksi

E_{cp} = $[33{,}000(0.150)^{1.5}\sqrt{5.892}\,]$ = 4,653.53 ksi

$n = \dfrac{4{,}653.53}{3834.25} = 1.214$

A = Area of girder cross section (non-composite section) = 788.4 in.^{2}

I = Moment of inertia about the centroid of the non-composite precast girder = 260,403 in.^{4}

e_g = Distance between centers of gravity of the girder and slab, in.
= $(t_s/2 + y_t)$ = (8/2 + 29.25) = 33.25 in.

$K_g = (1.214)[260403 + 788.4\,(33.25)^2] = 1{,}374{,}282.6$ in.^{4}

The approximate live load moment distribution factors for type k bridge girders, specified by LRFD Table 4.6.2.2.2b-1 are applicable if the following condition for K_g is satisfied (other requirements are provided in section A.2.5.2.2.1)

$10{,}000 \le K_g \le 7{,}000{,}000$

$10{,}000 \le 1{,}374{,}282.6 \le 7{,}000{,}000$ (O.K.)

For one design lane loaded:

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0\,L\,t_s^3}\right)^{0.1}$$

where:

DFM = Live load moment distribution factor for interior girders.

S = Spacing of adjacent girders = 8 ft.

L = Design span length = 108.583 ft.

t_s = Thickness of slab = 8 in.

$$DFM = 0.06 + \left(\frac{8}{14}\right)^{0.4}\left(\frac{8}{108.583}\right)^{0.3}\left(\frac{1,374,282.6}{12.0(108.583)(8)^3}\right)^{0.1}$$

DFM = 0.06 + (0.8)(0.457)(1.075) = 0.453 lanes/girder

For two or more lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,L\,t_s^3}\right)^{0.1}$$

$$DFM = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{108.583}\right)^{0.2}\left(\frac{1,374,282.6}{12.0(108.583)(8)^3}\right)^{0.1}$$

= 0.075 + (0.902)(0.593)(1.075) = 0.650 lanes/girder

The greater of the above two distribution factors governs. Thus, the case of two or more lanes loaded controls.

DFM = 0.650 lanes/girder

The live load moment distribution factor from Section A.2.5.2.2.1 is DFM = 0.639 lanes/girder

Percent difference in $DFM = \left(\frac{0.650 - 0.639}{0.650}\right)100 = 1.69\%$

The difference in the live load moment distribution factors is negligible and its impact on the live load moments will also be negligible. Hence, the live load moments obtained using distribution factor from Section A.2.5.2.2.1 can be used for the ultimate flexural strength design.

A.2.10 FATIGUE LIMIT STATE

LRFD Art. 5.5.3 specifies that the check for fatigue of the prestressing strands is not required for fully prestressed components that are designed to have extreme fiber tensile stress due to Service III limit state within the specified limit of $0.19\sqrt{f_c'}$.

The AASHTO Type IV girder in this design example is designed as a fully prestressed member and the tensile stress due to Service III limit state is less than $0.19\sqrt{f_c'}$ as shown in Section A.2.8.2.2. Hence, the fatigue check for the prestressing strands is not required.

A.2.11 FLEXURAL STRENGTH LIMIT STATE

[LRFD Art. 5.7.3]

The flexural strength limit state is investigated for Strength I load combination specified by LRFD Table 3.4.1-1 as follows

$M_u = 1.25(M_{DC}) + 1.5(M_{DW}) + 1.75(M_{LL+IM})$

where:

M_u = Factored ultimate moment at the midspan, k-ft.

M_{DC} = Moment at the midspan due to dead load of structural components and non-structural attachments, k-ft.
= $M_g + M_S + M_{barr}$

M_g = Moment at the midspan due to girder self-weight
= 1,209.98 k-ft.

M_S = Moment at the midspan due to slab weight
= 1,179.03 k-ft.

M_{barr} = Moment at the midspan due to barrier weight
= 160.64 k-ft.

M_{DC} = 1,209.98 + 1,179.03 + 160.64 = 2,549.65 k-ft.

M_{DW} = Moment at the midspan due to wearing surface load
= 188.64 k-ft.

M_{LL+IM} = Moment at the midspan due to vehicular live load including dynamic allowance, k-ft.
= $M_{LT} + M_{LL}$

M_{LT} = Distributed moment due to HS 20 truck load including dynamic load allowance = 1,423.00 k-ft.

M_{LL} = Distributed moment due to lane load = 602.72 k-ft.

M_{LL+IM} = 1,423.00 + 602.72 = 2,025.72 k-ft.

The factored ultimate bending moment at midspan

M_u = 1.25(2,549.65) + 1.5(188.64) + 1.75(2,025.72)
= 7,015.03 k-ft.

[LRFD Art. 5.7.3.1.1]

The average stress in the prestressing steel, f_{ps}, for rectangular or flanged sections subjected to flexure about one axis for which $f_{pe} \geq 0.5f_{pu}$, is given as:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{[LRFD Eq. 5.7.3.1.1-1]}$$

where:

f_{ps} = Average stress in the prestressing steel, ksi

f_{pu} = Specified tensile strength of prestressing steel = 270 ksi

f_{pe} = Effective prestress after final losses = $f_{pj} - \Delta f_{pT}$

f_{pj} = Jacking stress in the prestressing strands = 202.5 ksi

Δf_{pT} = Total final loss in prestress = 56.70 ksi (Section A.2.7.3.6)

f_{pe} = 202.5 – 56.70 = 145.80 ksi > $0.5f_{pu}$ = 0.5(270) = 135 ksi
Therefore, the equation for f_{ps} shown above is applicable.

k $= 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$ [LRFD Eq. 5.7.3.1.1-2]
= 0.28 for low-relaxation prestressing strands
[LRFD Table C5.7.3.1.1-1]

d_p = Distance from the extreme compression fiber to the centroid of the prestressing tendons, in.
= $h_c - y_{bs}$

h_c = Total height of the composite section = 54 + 8 = 62 in.

y_{bs} = Distance from centroid of the prestressing strands at midspan to the bottom fiber of the girder = 5.63 in. (see Section A.2.7.3.3)

d_p = 62 – 5.63 = 56.37 in.

c = Distance between neutral axis and the compressive face of the section, in.

The depth of neutral axis from the compressive face, c, is computed assuming rectangular section behavior. A check is made to confirm that the neutral axis is lying in the cast-in-place slab; otherwise the neutral axis will be calculated based on the flanged section behavior. [LRFD C5.7.3.2.2]

For rectangular section behavior,

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_s'}{0.85 f_c' \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1.-4]

A_{ps} = Area of prestressing steel, in.^{2}
= (number of strands)(area of each strand)
= 54(0.153) = 8.262 in.^{2}

f_{pu} = Specified tensile strength of prestressing steel = 270 ksi

A_s = Area of mild steel tension reinforcement = 0 in.^{2}

A_s' = Area of compression reinforcement = 0 in.^{2}

f_c' = Compressive strength of deck concrete = 4.0 ksi

f_y = Yield strength of tension reinforcement, ksi

f_y' = Yield strength of compression reinforcement, ksi

β_1 = Stress factor for compression block [LRFD Art. 5.7.2.2]

= 0.85 for $f_c' \leq 4.0$ ksi

b = Effective width of compression flange = 96 in. (based on non-transformed section)

Depth of neutral axis from compressive face

$$c = \frac{8.262(270) + 0 - 0}{0.85(4.0)(0.85)(96) + 0.28(8.262)\left(\dfrac{270}{56.37}\right)}$$

= 7.73 in. < t_s = 8.0 in. (O.K.)

The neutral axis lies in the slab, therefore the assumption of rectangular section behavior is valid.

The average stress in prestressing steel

$$f_{ps} = 270\left(1 - 0.28\frac{7.73}{56.37}\right) = 259.63 \text{ ksi}$$

For prestressed concrete members having rectangular section behavior, the nominal flexural resistance is given as:
[LRFD Art. 5.7.3.2.3]

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right)$$ [LRFD Eq. 5.7.3.2.2-1]

The above equation is a simplified form of LRFD Equation 5.7.3.2.2-1 because no compression reinforcement or mild tension reinforcement is provided.

a = Depth of the equivalent rectangular compression block, in.
= $\beta_1 c$

β_1 = Stress factor for compression block = 0.85 for $f'_c \leq 4.0$ ksi

a = 0.85(7.73) = 6.57 in.

Nominal flexural resistance

$$M_n = (8.262)(259.63)\left(56.37 - \frac{6.57}{2}\right)$$

= 113,870.67 k-in. = 9,489.22 k-ft.

Factored flexural resistance:

$$M_r = \phi M_n$$ [LRFD Eq. 5.7.3.2.1-1]

where:

ϕ = Resistance factor [LRFD Art. 5.5.4.2.1]
= 1.0 for flexure and tension of prestressed concrete members

M_r = 1*(9489.22) = 9,489.22 k-ft. > M_u = 7,015.03 k-ft. (O.K.)

A.2.12 LIMITS FOR REINFORCEMENT

[LRFD Art. 5.7.3.3]

A.2.12.1 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

The maximum amount of the prestressed and non-prestressed reinforcement should be such that

$$\frac{c}{d_e} \leq 0.42$$ [LRFD Eq. 5.7.3.3.1-1]

in which:

$$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y}$$ [LRFD Eq. 5.7.3.3.1-2]

c = Distance from the extreme compression fiber to the neutral axis = 7.73 in.

d_e = The corresponding effective depth from the extreme fiber to the centroid of the tensile force in the tensile reinforcement, in.
= d_p, if mild steel tension reinforcement is not used

d_p = Distance from the extreme compression fiber to the centroid of the prestressing tendons = 56.37 in.

Therefore d_e = 56.37 in.

$$\frac{c}{d_e} = \frac{7.73}{56.37} = 0.137 << 0.42 \qquad \text{(O.K.)}$$

A.2.12.2 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

At any section of a flexural component, the amount of prestressed and non-prestressed tensile reinforcement should be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

- 1.2 times he cracking moment, M_{cr}, determined on the basis of elastic stress distribution and the modulus of rupture of concrete, f_r
- 1.33 times the factored moment required by the applicable strength load combination.

The above requirements are checked at the midspan section in this design example. Similar calculations can be performed at any section along the girder span to check these requirements.

The cracking moment is given as

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \le S_c f_r \qquad \text{[LRFD Eq. 5.7.3.3.2-1]}$$

where:

f_r = Modulus of rupture, ksi
= $0.24\sqrt{f_c'}$ for normal weight concrete [LRFD Art. 5.4.2.6]

f_c' = Compressive strength of girder concrete at service
= 5.892 ksi

f_r = $0.24\sqrt{5.892}$ = 0.582 ksi

f_{cpe} = Compressive stress in concrete due to effective prestress force at extreme fiber of section where tensile stress is caused by externally applied loads, ksi

$= \dfrac{P_{pe}}{A} + \dfrac{P_{pe}e_c}{S_b}$

P_{pe} = Effective prestressing force after allowing for final prestress loss, kips
= (number of strands)(area of each strand)(f_{pe})
= 54(0.153)(145.80) = 1,204.60 kips

(Calculations for effective final prestress (f_{pe}) are shown in Section A.2.7.3.6)

e_c = Eccentricity of prestressing strands at the midspan
= 19.12 in.

A = Area of girder cross-section = 788.4 in.^{2}

S_b = Section modulus of the precast girder referenced to the extreme bottom fiber of the non-composite precast girder
= 10,521.33 in.^{3}

$$f_{cpe} = \frac{1,204.60}{788.4} + \frac{1,204.60(19.12)}{10,521.33}$$
= 1.528 + 2.189 = 3.717 ksi

M_{dnc} = Total unfactored dead load moment acting on the non-composite section
= $M_g + M_S$

M_g = Moment at the midspan due to girder self-weight
= 1,209.98 k-ft.

M_S = Moment at the midspan due to slab weight
= 1,179.03 k-ft.

M_{dnc} = 1,209.98 + 1,179.03 = 2,389.01 k-ft. = 28,668.12 k-in.

S_{nc} = Section modulus of the non-composite section referenced to the extreme fiber where the tensile stress is caused by externally applied loads = 10,521.33 in.^{3}

S_c = Section modulus of the composite section referenced to the extreme fiber where the tensile stress is caused by externally applied loads = 16,478.41 in.^{3} (based on updated composite section properties)

The cracking moment is:

$$M_{cr} = (16{,}478.41)(0.582 + 3.717) - (28{,}668.12)\left(\frac{16{,}478.41}{10{,}521.33} - 1\right)$$

$$= 70{,}840.68 - 16{,}231.62 = 54{,}609.06 \text{ k-in.} = 4{,}550.76 \text{ k-ft.}$$

$S_c f_r = (16{,}478.41)(0.582) = 9{,}590.43$ k-in.
$= 799.20$ k-ft. $< 4{,}550.76$ k-ft.

Therefore use $M_{cr} = 799.20$ k-ft.

$1.2\, M_{cr} = 1.2(799.20) = 959.04$ k-ft.

Factored moment required by Strength I load combination at midspan
$M_u = 7{,}015.03$ k-ft.

$1.33\, M_u = 1.33(7{,}015.03$ k-ft.$) = 9{,}330$ k-ft.

Since, $1.2\, M_{cr} < 1.33\, M_u$, the $1.2M_{cr}$ requirement controls.

$M_r = 9{,}489.22$ k-ft $>> 1.2\, M_{cr} = 959.04$ (O.K.)

A.2.13 TRANSVERSE SHEAR DESIGN

The area and spacing of shear reinforcement must be determined at regular intervals along the entire span length of the girder. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports. Similar calculations can be performed to determine shear reinforcement requirements at any selected section.

LRFD Art. 5.8.2.4 specifies that the transverse shear reinforcement is required when:

$$V_u < 0.5\,\phi\,(V_c + V_p) \qquad \text{[LRFD Art. 5.8.2.4-1]}$$

where:

V_u = Total factored shear force at the section, kips

V_c = Nominal shear resistance of the concrete, kips

V_p = Component of the effective prestressing force in the direction of the applied shear, kips

ϕ = Resistance factor = 0.90 for shear in prestressed concrete members [LRFD Art. 5.5.4.2.1]

A.2.13.1 Critical Section

Critical section near the supports is the greater of:

[LRFD Art. 5.8.3.2]

$$0.5\ d_v \cot\theta \text{ or } d_v$$

where:

d_v = Effective shear depth, in.
= Distance between the resultants of tensile and compressive forces, (d_e - a/2), but not less than the greater of (0.9d_e) or (0.72h) [LRFD Art. 5.8.2.9]

d_e = Corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement [LRFD Art. 5.7.3.3.1]

a = Depth of compression block = 6.57 in. at midspan (see Section A.2.11)

h = Height of composite section = 62 in.

A.2.13.1.1 Angle of Diagonal Compressive Stresses

The angle of inclination of the diagonal compressive stresses is calculated using an iterative method. As an initial estimate θ is taken as 23^0.

A.2.13.1.2 Effective Shear Depth

The shear design at any section depends on the angle of diagonal compressive stresses at the section. Shear design is an iterative process that begins with assuming a value for θ.

Because some of the strands are harped at the girder end, the effective depth d_e, varies from point to point. However d_e must be calculated at the critical section for shear which is not yet known. Therefore, for the first iteration, d_e is calculated based on the center of gravity of the straight strand group at the end of the girder, y_{bsend}. This methodology is given in *PCI Bridge Design Manual* (PCI 2003)

Effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement

$d_e = h - y_{bsend}$ = 62.0 – 5.55 = 56.45 in. (see Section A.2.7.3.9 for y_{bsend})

Effective shear depth

$$d_v = d_e - 0.5(a) = 56.45 - 0.5(6.57) = 53.17 \text{ in.} \quad \text{(controls)}$$

$$\geq 0.9d_e = 0.9(56.45) = 50.80 \text{ in.}$$

$$\geq 0.72h = 0.72(62) = 44.64 \text{ in.} \quad \text{(O.K.)}$$

Therefore d_v = 53.17 in.

A.2.13.1.3 Calculation of critical section

[LRFD Art. 5.8.3.2]

The critical section near the support is greater of:

d_v = 53.17 in. and

0.5 d_v cot θ = 0.5(53.17)(cot 23^0) = 62.63 in. from the face of the support (controls)

Adding half the bearing width (3.5 in., standard pad size for prestressed girders is 7" × 22") to the critical section distance from the face of the support to get the distance of the critical section from the centerline of bearing.

Critical section for shear

x = 62.63 + 3.5 = 66.13 in. = 5.51 ft. (0.051L) from the centerline of bearing where L is the design span length.

The value of d_e is calculated at the girder end which can be refined based on the critical section location. However, it is conservative not to refine the value of d_e based on the critical section 0.051L. The value if refined will have a small difference (PCI 2003).

A.2.13.2 Contribution of Concrete to Nominal Shear Resistance

[LRFD Art. 5.8.3.3]

The contribution of the concrete to the nominal shear resistance is given as:

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

where:

β = A factor indicating the ability of diagonally cracked concrete to transmit tension

f'_c = Compressive strength of concrete at service = 5.892 ksi

b_v = effective web width taken as the minimum web width within the depth d_v, in. = 8 in. (see Figure A.2.4.1)

d_v = Effective shear depth = 53.17 in.

A.2.13.2.1 Strain in Flexural Tension Reinforcement

[LRFD Art. 5.8.3.4.2]

The θ and β values are determined based on the strain in the flexural tension reinforcement. The strain in the reinforcement, ε_x is determined assuming that the section contains at least the minimum transverse reinforcement as specified in LRFD Art. 5.8.2.5

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \leq 0.001$$

[LRFD Eq. 5.8.3.4.2-1]

where:

V_u = Applied factored shear force at the specified section, $0.051L$
= 1.25(40.04 + 39.02 + 5.36) +1.50(6.15) + 1.75(67.28 + 25.48) = 277.08 kips

M_u = Applied factored moment at the specified section, 0.051L $> V_ud_v$
= 1.25(233.54 + 227.56 + 31.29) + 1.50(35.84) + 1.75(291.58 + 116.33)
= 1383.09 k-ft. > 277.08(53.17/12) = 1,227.69 k-ft. (O.K.)

N_u = Applied factored normal force at the specified section, $0.051L$ = 0 kips

f_{po} = Parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked in difference in strain between the prestressing tendons and the surrounding concrete (ksi) For pretensioned members, LRFD Art. C5.8.3.4.2 indicates that f_{po} can be taken as the stress in strands when the concrete is cast around them, which is jacking stress f_{pj}, or f_{pu}.
= 0.75(270.0) = 202.5 ksi

V_p = Component of the effective prestressing force in the direction of the applied shear, kips
= (Force per strand)(Number of harped strands)(sinΨ)

$$\Psi = \tan^{-1}\left(\frac{42.45}{49.4(12\text{in./ft.})}\right) = 0.072 \text{ rad. (see Figure A.2.7.3)}$$

V_p = 22.82(10) sin (0.072) = 16.42 kips

$$\varepsilon_x = \frac{\frac{1383.09(12\text{ in./ft.})}{53.17} + 0.5(277.08 - 16.42)\cot 23^\circ - 44(0.153)202.5}{2[28000(0.0) + 28500(44)(0.153)]}$$

$\varepsilon_x = -0.00194$

Since this value is negative LRFD Eq. 5.8.3.4.2-3 should be used to calculate ε_x

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})}$$

where:

A_c = Area of the concrete on the flexural tension side below $h/2 = 473$ in.^{2}

E_c = Modulus fo elasticity of girder concrete, ksi
= $33{,}000(w_c)^{1.5}\sqrt{f_c'}$
= $[33{,}000(0.150)^{1.5}\sqrt{5.892}\,] = 4{,}653.53$ ksi

Strain in the flexural tension reinforcement is

$$\varepsilon_x = \frac{\frac{1383.09(12\text{ in./ft.})}{53.17} + 0.5(277.08 - 16.42)\cot 23^\circ - 44(0.153)202.5}{2[4653.53(473) + 28000(0.0) + 28500(44)(0.153)]}$$

$\varepsilon_x = -0.000155$

Shear stress in the concrete is given as

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ [LRFD Eq. 5.8.3.4.2-1]

where:

ϕ = Resistance factor = 0.9 for shear in prestressed concrete members [LRFD Art. 5.5.4.2.1]

$$v_u = \frac{277.08 - 0.9(16.42)}{0.9(8.0)(53.17)} = 0.685 \text{ ksi}$$

$\upsilon_u / f_c' = 0.685/5.892 = 0.12$

A.2.13.2.2 Values of β and θ

The values of β and θ are determined using LRFD Table 5.8.3.4.2-1. Linear interpolation is allowed if the values lie between two rows

Table A.2.13.1. Interpolation for θ and β Values

υ_u / f'_c	ε_x x 1000		
	≤-0.200	-0.155	≤-0.100
≤0.100	18.100		20.400
	3.790		3.380
0.12	19.540	20.47	21.600
	3.302	3.20	3.068
≤0.125	19.900		21.900
	3.180		2.990

$\theta = 20.47^0 > 23^0$ (assumed)

Another iteration is made with $\theta = 20.65^0$ to arrive at the correct value of β and θ.

d_e = Effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement = 56.45 in.

d_v = Effective shear depth = 53.17 in.

The critical section near the support is greater of:
d_v = 53.17 in. and

$0.5d_v\cot\theta = 0.5(53.17)(\cot 20.47^0) = 71.2$ in. from the face of the support (controls)

Add half the bearing width (3.5 in.) to critical section distance from the face of the support to get the distance of the critical section from centerline of bearing.

Critical section for shear

$x = 71.2 + 3.5 = 74.7$ in. = 6.22 ft. ($0.057L$) from the centerline of bearing

Assuming the strain will be negative again, LRFD Eq. 5.8.3.4.2-3 will be used to calculate ε_x

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})}$$

The shear forces and bending moments will be updated based on the updated critical section location.

V_u = Applied factored shear force at the specified section, $0.057L$
= 1.25(39.49 + 38.48 + 5.29) +1.50(6.06) + 1.75(66.81 + 25.15) = 274.10 kips

M_u = Applied factored moment at the specified section, $0.057L > V_u d_v$
= 1.25(260.18 + 253.53 + 34.86) + 1.50(39.93) + 1.75(324.63 + 129.60)
= 1540.50 k-ft. > 274.10(53.17/12) = 1222.03 k-ft.(O.K.)

$$\varepsilon_x = \frac{\frac{1540.50(12\text{ in./ft.})}{53.17} + 0.5(274.10 - 16.42)\cot 20.47^\circ - 44(0.153)202.5}{2[4653.53(473) + 28000(0.0) + 28500(44)(0.153)]}$$

$\varepsilon_x = -0.000140$

Shear stress in concrete

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{274.10 - 0.9(16.42)}{0.9(8)(53.17)} = 0.677 \text{ ksi}$$

[LRFD Eq. 5.8.3.4.2-1]

$\upsilon_u / f'_c = 0.677/5.892 = 0.115$

Table A.2.13.2. Interpolation for θ and β Values

υ_u / f'_c	ε_x x 1000		
	≤–0.200	–0.140	≤–0.100
≤0.100	18.100		20.40
	3.790		3.380
0.115	18.59	20.22	21.30
	3.424	3.26	3.146
≤0.125	19.90		21.900
	3.180		2.990

$\theta = 20.22^0 \approx 20.47^0$ (from first iteration)

Therefore no further iteration is needed.

$\beta = 3.26$

A.2.13.2.3 Computation of Concrete Contribution

The contribution of the concrete to the nominal shear resistance is given as:

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

where:

β = A factor indicating the ability of diagonally cracked concrete to transmit tension = 3.26

f'_c = Compressive strength of concrete at service = 5.892 ksi

b_v = effective web width taken as the minimum web width within the depth d_v, in. = 8 in. (see Figure A.2.4.1)

d_v = Effective shear depth = 53.17 in.

$V_c = 0.0316(3.26)(\sqrt{5.892}\ (8.0)(53.17) = 106.36$

A.2.13.3 Contribution of Reinforcement to Nominal Shear Resistance

A.2.11.3.1 Requirement for Reinforcement

Check if $V_u > 0.5\,\phi\,(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

V_u = 274.10 kips > 0.5(0.9)(106.36 + 16.42) = 55.25 kips

Therefore, transverse shear reinforcement should be provided.

A.2.13.3.2 Required Area of Reinforcement

The required area of transverse shear reinforcement is

$$\frac{V_u}{\phi} \le V_n = (V_c + V_s + V_p) \qquad \text{[LRFD Eq. 5.8.3.3-1]}$$

where

V_s = Shear force carried by transverse reinforcement.

$$= \frac{V_u}{\phi} - V_c - V_p = \left(\frac{274.10}{0.9} - 106.36 - 16.42\right) = 181.77 \text{ kips}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$ [LRFD Eq. 5.8.3.3-4]

where

A_v = Area of shear reinforcement within a distance s, in.^{2}

s = Spacing of stirrups, in.

f_y = Yield strength of shear reinforcement, ksi

α = angle of inclination of transverse reinforcement to longitudinal axis = 90^0 for vertical stirrups

Therefore, area of shear reinforcement within a distance s is:

$A_v = (sV_s)/ f_y d_v(\cot\theta + \cot\alpha)\sin\alpha$

$= s(181.77)/(60)(53.17)(\cot 20.22^0 + \cot 90^0) \sin 90^0 = 0.021(s)$

If s = 12 in., required $A_v = 0.252$ in^2 / ft

A.2.13.3.3 Determine spacing of reinforcement

Check for maximum spacing of transverse reinforcement [LRFD Art.. 5.8.2.7]

check if $v_u < 0.125 f_c'$ [LRFD Eq. 5.8.2.7-1]

or if $v_u \geq 0.125 f_c'$ [LRFD Eq. 5.8.2.7-2]

$0.125 f_c' = 0.125(5.892) = 0.74$ ksi

$v_u = 0.677$ ksi

Since $v_u < 0.125 f_c'$, Therefore, $s \leq 24$ in. [LRFD Eq. 5.8.2.7-2]

$s \leq 0.8\, d_v = 0.8(53.17) = 42.54$ in.

Therefore maximum s = 24.0 in. > s provided (O.K.)

Use #4 bar double legged stirrups at 12 in. c/c,

$A_v = 2(0.20) = 0.40$ in^2/ft > 0.252 in^2/ft

$$V_s = \frac{0.4(60)(53.17)(\cot 20.47^0)}{12} = 283.9 \text{ kips}$$

A.2.11.3.4 Minimum Reinforcement requirement

The area of transverse reinforcement should not be less than:

[LRFD Art. 5.8.2.5]

$$0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y}$$ [LRFD Eq. 5.8.2.5-1]

$$= 0.0316\sqrt{5.892}\,\frac{(8)(12)}{60} = 0.12 < A_v \text{ provided} \qquad \text{(O.K.)}$$

A.2.13.5 Maximum Nominal Shear Resistance

In order to assure that the concrete in the web of the girder will not crush prior to yield of the transverse reinforcement, the LRFD Specifications give an upper limit for V_n as follows:

$$V_n = 0.25\ f'_c\, b_v d_v + V_p$$ [LRFD Eq. 5.8.3.3-2]

Comparing above equation with LRFD Eq. 5.8.3.3-1

$$V_c + V_s \leq 0.25\ f'_c\, b_v d_v$$

$$106.36 + 283.9 = 390.26 \text{ kips} \leq 0.25(5.892)(8)(53.17)$$
$$= 626.55 \text{ kips} \qquad \text{O.K.}$$

This is a sample calculation for determining transverse reinforcement requirement at critical section and this procedure can be followed to find the transverse reinforcement requirement at increments along the length of the girder.

A.2.14 INTERFACE SHEAR TRANSFER

A.2.12.1 Factored Horizontal Shear

[LRFD Art. 5.8.4]

At the strength limit state, the horizontal shear at a section can be calculated as follows

$$V_h = \frac{V_u}{d_v}$$ [LRFD Eq. C5.8.4.1-1]

where

V_h = Horizontal shear per unit length of the girder, kips

V_u = Factored shear force at specified section due to superimposed loads, kips

d_v = Distance between resultants of tensile and compressive forces (d_e-a/2), in.

The LRFD Specifications do not identify the location of the critical section. For convenience, it will be assumed here to be the same location as the critical section for vertical shear, at point 0.057L

Using load combination Strength I:

V_u = 1.25(5.29) +1.50(6.06) + 1.75(66.81 + 25.15) = 176.63 kips

d_v = 53.17 in

Therefore applied factored horizontal shear is:

$$V_h = \frac{176.63}{53.17} = 3.30 \text{ kips/in.}$$

Required $V_n = V_h / \varphi = 3.30/0.9 = 3.67$ kip/in

A.2.14.2 Required Nominal Resistance

The nominal shear resistance of the interface surface is:

$$V_n = cA_{cv} + \mu\,[A_{vf} f_y + P_c]$$ [LRFD Eq. 5.8.4.1-1]

where

c = Cohesion factor [LRFD Art. 5.8.4.2]

μ = Friction factor [LRFD Art. 5.8.4.2]

A_{cv} = Area of concrete engaged in shear transfer, in^2.

A_{vf} = Area of shear reinforcement crossing the shear plane, in^2.

P_c = Permanent net compressive force normal to the shear plane, kips

f_y = Shear reinforcement yield strength, ksi

A.2.14.3 Required Interface Shear Reinforcement

For concrete placed against clean, hardened concrete and free of laitance, but not an intentionally roughened surface:

[LRFD Art. 5.8.4.2]

c = 0.075 ksi

$\mu = 0.6\lambda$, where $\lambda = 1.0$ for normal weight concrete, and therefore,

$\mu = 0.6$

The actual contact width, b_v, between the slab and the girder is 20 in.

A_{cv} = (20 in.)(1 in) = 20 in.^{2}

The LRFD Eq. 5.8.4.1-1 can be solved for A_{vf} as follows:

$3.67 = (0.075)(20) + 0.6(A_{vf}(60) + 0)$

Solving for A_{vf} = 0.06 in^2/in or 0.72 in.^{2} / ft.

2 - #4 double-leg bar per ft are provided.

Area of steel provided = 2 (0.40) = 0.80 in.^{2} / ft.

Provide 2 legged #4 bars at 6 in. c/c

The web reinforcement shall be provided at 6 in. c/c which can be extended into the cast-in-place slab to account for the interface shear requirement.

A.2.14.3.1 Minimum Interface shear reinforcement

Minimum $A_{vf} \geq (0.05b_v)/f_y$ [LRFD Eq. 5.8.4.1-4]

where b_v = width of the interface

A_{vf} = 0.80 in.^{2}/ft. > [0.05(20)/60](12 in./ft) = 0.2 in.^{2}/ft. O.K.

$$V_n \text{ provided} = 0.075(20) + 0.6\left(\frac{0.80}{12}(60) + 0\right) = 3.9 \text{ kips/in.}$$

$0.2\ f'_c A_{cv} = 0.2(4.0)(20) = 16$ kips/in.

$0.8A_{cv} = 0.8(20) = 16$ kips/in.

Since provided $V_n \leq 0.2\ f'_c A_{cv}$ O.K. [LRFD Eq. 5.8.4.1-2]

$\leq 0.8A_{cv}$ O.K. [LRFD Eq. 5.8.4.1-3]

A.2.15 MINIMUM LONGITUDINAL REINFORCEMENT REQUIREMENT

[LRFD Art. 5.8.3.5]

Longitudinal reinforcement should be proportioned so that at each section the following equation is satisfied

$$A_s f_y + A_{ps} f_{ps} \geq \frac{M_u}{d_v \phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

[LRFD Eq. 5.8.3.5-1]

where

A_s = Area of non prestressed tension reinforcement, in.^{2}

f_y = Specified minimum yield strength of reinforcing bars, ksi

A_{ps} = Area of prestressing steel at the tension side of the section, in.^{2}

f_{ps} = Average stress in prestressing steel at the time for which the nominal resistance is required, ksi

M_u = Factored moment at the section corresponding to the factored shear force, kip-ft.

N_u = Applied factored axial force, kips

V_u = Factored shear force at the section, kips

V_s = Shear resistance provided by shear reinforcement, kips

V_p = Component in the direction of the applied shear of the effective prestressing force, kips

d_v = Effective shear depth, in.

θ = Angle of inclination of diagonal compressive stresses.

A.2.15.1 Required Reinforcement at Face of Bearing

[LRFD Art. 5.8.3.5]

Width of bearing = 7.0 in.

Distance of section = 7/2 = 3.5 in. = 0.291 ft.

Shear forces and bending moment are calculated at this section

V_u = 1.25(44.35 + 43.22 + 5.94) + 1.50(6.81) + 1.75(71.05 + 28.14)

= 300.69 kips.

M_u = 1.25(12.04 + 11.73 + 1.61) + 1.50(1.85) + 1.75(15.11 + 6.00)

= 71.44 Kip-ft.

$$\frac{M_u}{d_v \phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

$$= \frac{71.44(12 \text{ in./ft.})}{53.17(0.9)} + 0 + \left(\frac{300.69}{0.90} - 0.5(283.9) - 16.42\right)\cot 20.47^0$$

= 484.09 kips

The crack plane crosses the centroid of the 44 straight strands at a distance of 6 + 5.33 cot 20.47^0 = 20.14 in. from the end of the girder.

Since the transfer length is 30 in. the available prestress from 44 straight strands is a fraction of the effective prestress, f_{pe}, in these strands. The 10 harped strands do not contribute the tensile capacity since they are not on the flexural tension side of the member.

Therefore available prestress force is:

$$A_s f_y + A_{ps} f_{ps} = 0 + 44(0.153)\left(149.18\frac{20.33}{30}\right) = 680.57 \text{ kips}$$

$A_s f_y + A_{ps} f_{ps}$ = 649.63 kips > 484.09 kips

Therefore additional longitudinal reinforcement is not required.

A.2.16 PRETENSIONED ANCHORAGE ZONE

[LRFD Art. 5.10.10]

A.2.16.1 Minimum Vertical Reinforcement

[LRFD Art. 5.10.10.1]

Design of the anchorage zone reinforcement is computed using the force in the strands just prior to transfer:

Force in the strands at transfer
F_{pi} = 54(0.153)(202.5) = 1673.06 kips

The bursting resistance, P_r, should not be less than 4% of F_{pi}
[LRFD Arts. 5.10.10.1 and C3.4.3]

$P_r = f_sA_s \geq 0.04F_{pi} = 0.04(1673.06) = 66.90$ kips

where

A_s = Total area of vertical reinforcement located within a distance of h/4 from the end of the girder, in ^{2}.

f_s = Stress in steel not exceeding 20 ksi.

Solving for required area of steel A_s= 66.90/20 = 3.35 in^2

Atleast 3.35 in^2 of vertical transverse reinforcement should be provided within a distance of (h/4 = 62 / 4 = 15.5 in). from the end of the girder.

Use 6 - #5 double leg bars at 2.0 in. spacing starting at 2 in. from the end of the girder.

The provided A_s = 6(2)0.31 = 3.72 in^2 > 3.35 in^2 O.K.

A.2.16.2 Confinement Reinforcement

[LRFD Art. 5.10.10.2]

For a distance of 1.5d = 1.5(54) = 81 in. from the end of the girder, reinforcement is placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than #3 deformed bars with spacing not exceeding 6 in. The reinforcement should be of shape which will confine the strands.

A.2.17 CAMBER AND DEFLECTIONS

A.2.17.1 Maximum Camber

The LRFD Specifications do not provide any guidelines for the determination of camber of prestressed concrete members. The Hyperbolic Functions Method proposed by Rauf and Furr (1970) for the calculation of maximum camber is used by TxDOT's prestressed concrete bridge design software, PSTRS14 (TxDOT 2004). The following steps illustrate the Hyperbolic Functions method for the estimation of maximum camber.

Step 1: The total prestressing force after initial prestress loss due to elastic shortening has occurred

$$P = \frac{P_i}{\left(1 + pn + \frac{e_c^2 A_s n}{I}\right)} + \frac{M_D\, e_c\, A_s\, n}{I\left(1 + pn + \frac{e_c^2 A_s n}{I}\right)}$$

where:

P_i = Anchor force in prestressing steel
= (number of strands)(area of strand)(f_{si})

P_i = 54(0.153)(202.5) = 1673.06 kips

f_{pi} = Before transfer, $\leq$ 0.75 f_{pu} = 202,500 psi

[LRFD Table 5.9.3-1]

f_{pu} = Ultimate strength of prestressing strands = 270 ksi

f_{pi} = 0.75(270) = 202.5 ksi

I = Moment of inertia of the non-composite precast girder
= 260403 in.^{4}

e_c = Eccentricity of prestressing strands at the midspan
= 19.12 in.

M_D = Moment due to self-weight of the girder at midspan
= 1209.98 k-ft.

A_s = Area of prestressing steel
= (number of strands)(area of strand)
= 54(0.153) = 8.262 in.^{2}

p = A_s/A

A = Area of girder cross-section = 788.4 in.^{2}

$$p = \frac{8.262}{788.4} = 0.0105$$

n = Modular ratio between prestressing steel and the girder concrete at release = E_s/E_{ci}

E_{ci} = Modulus of elasticity of the girder concrete at release
= $33(w_c)^{3/2}\sqrt{f'_{ci}}$ [STD Eq. 9-8]

w_c = Unit weight of concrete = 150 pcf

f'_{ci} = Compressive strength of precast girder concrete at release = 5,892 psi

$$E_{ci} = [33(150)^{3/2}\sqrt{5{,}892}\,]\left(\frac{1}{1{,}000}\right) = 4{,}653.53 \text{ ksi}$$

E_s = Modulus of elasticity of prestressing strands
= 28,000 ksi

n = 28,500/4,653.53 = 6.12

$$\left(1 + pn + \frac{e_c^2 A_s n}{I}\right) = 1 + (0.0105)(6.\ 12) + \frac{(19.12^2)(8.262)(6.12)}{260{,}403}$$
$$= 1.135$$

$$P = \frac{1{,}673.06}{1.135} + \frac{(1{,}209.98)(12 \text{ in./ft.})(19.12)(8.262)(6.12)}{260{,}403(1.135)}$$

$$= 1474.06 + 47.49 = 1521.55 \text{ kips}$$

Initial prestress loss is defined as

$$PL_i = \frac{P_i - P}{P_i} = \frac{1{,}673.06 - 1521.55}{1{,}673.06} = 0.091 = 9.1\%$$

The stress in the concrete at the level of the centroid of the prestressing steel immediately after transfer is determined as follows.

$$f_{ci}^{s} = P\left(\frac{1}{A} + \frac{e_c^2}{I}\right) - f_c^{s}$$

where:

f_c^s = Concrete stress at the level of centroid of prestressing steel due to dead loads, ksi

$$= \frac{M_D\, e_c}{I} = \frac{(1{,}209.98)(12 \text{ in./ft.})(19.12)}{260{,}403} = 1.066 \text{ ksi}$$

$$f_{ci}^{s} = 1521.55\left(\frac{1}{788.4} + \frac{19.12^2}{260{,}403}\right) - 1.066 = 3.0 \text{ ksi}$$

The ultimate time dependent prestress loss is dependent on the ultimate creep and shrinkage strains. As the creep strains vary with the concrete stress, the following steps are used to evaluate the concrete stresses and adjust the strains to arrive at the ultimate prestress loss. It is assumed that the creep strain is proportional to the concrete stress and the shrinkage stress is independent of concrete stress. (Sinno 1970)

Step 2: Initial estimate of total strain at steel level assuming constant sustained stress immediately after transfer

$$\varepsilon_{c1}^{s} = \varepsilon_{cr}^{\infty} f_{ci}^{s} + \varepsilon_{sh}^{\infty}$$

where:

$\varepsilon_{cr}^{\infty}$ = Ultimate unit creep strain = 0.00034 in./in. [this value is prescribed by Sinno et. al. (1970)]

$\varepsilon_{sh}^{\infty}$ = Ultimate unit shrinkage strain = 0.000175 in./in. [this value is prescribed by Sinno et. al. (1970)]

$\varepsilon_{c1}^{s} = 0.00034(3.0) + 0.000175 = 0.001195$ in./in.

Step 3: The total strain obtained in Step 2 is adjusted by subtracting the elastic strain rebound as follows

$$\varepsilon_{c2}^{s} = \varepsilon_{c1}^{s} - \varepsilon_{c1}^{s} E_s \frac{A_s}{E_{ci}}\left(\frac{1}{A} + \frac{e_c^2}{I}\right)$$

$$\varepsilon_{c2}^{s} = 0.001195 - 0.001195\ (28{,}500)\frac{8.262}{4{,}653.53}\left(\frac{1}{788.4} + \frac{19.12^2}{260{,}403}\right)$$

= 0.001033 in./in.

Step 4: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_c^{s} = \varepsilon_{c2}^{s} E_s A_s \left(\frac{1}{A} + \frac{e_c^2}{I}\right)$$

$$\Delta f_c^{s} = 0.001033\ (28{,}500)(8.262)\left(\frac{1}{788.4} + \frac{19.12^2}{260{,}403}\right) = 0.648 \text{ ksi}$$

Step 5: The total strain computed in Step 2 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\varepsilon_{c4}^{s} = \varepsilon_{cr}^{\infty}\left(f_{ci}^{s} - \frac{\Delta f_c^{s}}{2}\right) + \varepsilon_{sh}^{\infty}$$

$$\varepsilon_{c4}^{s} = 0.00034\left(3.0 - \frac{0.648}{2}\right) + 0.000175 = 0.001085 \text{ in./in.}$$

Step 6: The total strain obtained in Step 5 is adjusted by subtracting the elastic strain rebound as follows

$$\varepsilon_{c5}^{s} = \varepsilon_{c4}^{s} - \varepsilon_{c4}^{s} E_s \frac{A_s}{E_{ci}}\left(\frac{1}{A} + \frac{e_c^2}{I}\right)$$

$$\varepsilon_{c5}^{s} = 0.001085 - 0.001085(28500)\ \frac{8.262}{4653.53}\left(\frac{1}{788.4} + \frac{19.12^2}{260403}\right)$$

= 0.000938 in./in

Sinno (1970) recommends stopping the updating of stresses and adjustment process after Step 6. However, as the difference between the strains obtained in Steps 3 and 6 is not negligible, this process is carried on until the total strain value converges.

Step 7: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_{c1}^{s} = \varepsilon_{c5}^{s} E_s A_s \left(\frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\Delta f_{c1}^{s} = 0.000938(28{,}500)(8.262)\left(\frac{1}{788.4} + \frac{19.12^2}{260{,}403} \right) = 0.5902 \text{ ksi}$$

Step 8: The total strain computed in Step 5 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\varepsilon_{c6}^{s} = \varepsilon_{cr}^{\infty} \left(f_{ci}^{s} - \frac{\Delta f_{c1}^{s}}{2} \right) + \varepsilon_{sh}^{\infty}$$

$$\varepsilon_{c6}^{s} = 0.00034\left(3.0 - \frac{0.5902}{2} \right) + 0.000175 = 0.001095 \text{ in./in.}$$

Step 9: The total strain obtained in Step 8 is adjusted by subtracting the elastic strain rebound as follows

$$\varepsilon_{c7}^{s} = \varepsilon_{c6}^{s} - \varepsilon_{c6}^{s} E_s \frac{A_s}{E_{ci}} \left(\frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\varepsilon_{c7}^{s} = 0.001095 - 0.001095(28{,}500)\,\frac{8.262}{4{,}653.53}\left(\frac{1}{788.4} + \frac{19.12^2}{260{,}403} \right)$$

$$= 0.000947 \text{ in./in}$$

The strains have sufficiently converged and no more adjustments are needed.

Step 10: Computation of final prestress loss

Time dependent loss in prestress due to creep and shrinkage strains is given as

$$PL^{\infty} = \frac{\varepsilon_{c7}^{s} E_s A_s}{P_i} = \frac{0.000947(28{,}500)(8.262)}{1{,}673.06} = 0.133 = 13.3\%$$

Total final prestress loss is the sum of initial prestress loss and the time dependent prestress loss expressed as follows

$$PL = PL_i + PL^{\infty}$$

where:

PL = Total final prestress loss percent.

PL_i = Initial prestress loss percent = 9.1%

PL^{∞}= Time dependent prestress loss percent = 13.3%

PL = 9.1 + 13.3 = 22.4%

Step 11: The initial deflection of the girder under self-weight is calculated using the elastic analysis as follows:

$$C_{DL} = \frac{5\,w\,L^4}{384 E_{ci} I}$$

where:

C_{DL} = Initial deflection of the girder under self-weight, ft.

w = Self-weight of the girder = 0.821 kips/ft.

L = Total girder length = 109.67 ft.

E_{ci} = Modulus of elasticity of the girder concrete at release = 4,653.53 ksi = 670,108.32 k/ft.^{2}

I = Moment of inertia of the non-composite precast girder = 260403 in.^{4} = 12.558 ft.^{4}

$$C_{DL} = \frac{5(0.821)(109.67^4)}{384(670{,}108.32)(12.558)} = 0.184 \text{ ft.} = 2.208 \text{ in.}$$

Step 12: Initial camber due to prestress is calculated using the moment area method. The following expression is obtained from the M/EI diagram to compute the camber resulting from the initial prestress.

$$C_{pi} = \frac{M_{pi}}{E_{ci} I}$$

where:

M_{pi} = $[0.5(P)\,(e_e)\,(0.5L)^2 + 0.5(P)\,(e_c - e_e)\,(0.67)\,(HD)^2$
$+0.5P\,(e_c - e_e)\,(HD_{dis})\,(0.5L + HD)]/(Eci)(I)$

P = Total prestressing force after initial prestress loss due to elastic shortening have occurred = 1521.55 kips

HD = Hold down distance from girder end
= 49.404 ft. = 592.85 in. (see Figure A.1.7.3)

HD_{dis} = Hold down distance from the center of the girder span
= 0.5(109.67) – 49.404 = 5.431 ft. = 65.17 in.

e_e = Eccentricity of prestressing strands at girder end
= 11.34 in.

e_c = Eccentricity of prestressing strands at midspan
= 19.12 in.

L = Overall girder length = 109.67 ft. = 1,316.04 in.

$M_{pi} = \{0.5(1521.55)\,(11.34)\,[(0.5)\,(1{,}316.04)]^2 +$
$0.5(1521.55\,)\,(19.12 - 11.34)\,(0.67)\,(592.85)^2 +$
$0.5(1521.55\,)\,(19.12 - 11.34)\,(65.17)[0.5(1316.04) +$
$592.85]\}$

$M_{pi} = 3.736 \text{ x } 10^9 + 1.394 \text{ x } 10^9 + 0.483 \text{ x } 10^9 = 5.613 \text{ x } 10^9$

$$C_{pi} = \frac{5.613\times10^9}{(4{,}653.53)(260{,}403)} = 4.63 \text{ in.} = 0.386 \text{ ft.}$$

Step 13: The initial camber, C_I, is the difference between the upward camber due to initial prestressing and the downward deflection due to self-weight of the girder.

$$C_i = C_{pi} - C_{DL} = 4.63 - 2.208 = 2.422 \text{ in.} = 0.202 \text{ ft.}$$

Step 14: The ultimate time-dependent camber is evaluated using the following expression.

$$\text{Ultimate camber } C_t = C_i\,(1 - PL^{\infty})\frac{\varepsilon_{cr}^{\infty}\left(f_{ci}^{s} - \dfrac{\Delta f_{c1}^{s}}{2}\right) + \varepsilon_e^{s}}{\varepsilon_e^{s}}$$

where:

$$\varepsilon_e^{s} = \frac{f_{ci}^{s}}{E_{ci}} = \frac{3.0}{4{,}653.53} = 0.000619 \text{ in./in.}$$

$$C_t = 2.422(1 - 0.133)\frac{0.00034\left(3.0 - \dfrac{0.5902}{2}\right) + 0.000645}{0.000645}$$

$C_t = 5.094$ in. = 0.425 ft. ↑

A.2.17.2 Deflection Due to Slab Weight

The deflection due to the slab weight is calculated using an elastic analysis as follows.

Deflection of the girder at midspan

$$\Delta_{slab1} = \frac{5\,w_s\,L^4}{384\,E_c\,I}$$

where:

w_s = Weight of the slab = 0.80 kips/ft.

E_c = Modulus of elasticity of girder concrete at service
$= 33(w_c)^{3/2}\sqrt{f_c'}$
$= 33(150)^{1.5}\sqrt{5{,}892}\left(\dfrac{1}{1{,}000}\right) = 4{,}653.53$ ksi

I = Moment of inertia of the non-composite girder section
= 260,403 in.^{4}

L = Design span length of girder (center to center bearing)
= 108.583 ft.

$$\Delta_{slab1} = \frac{5\left(0.80/12\text{ in./ft.}\right)[(108.583)(12\text{ in./ft.})]^4}{384(4{,}653.53)(260{,}403)}$$

= 2.06 in. = 0.172 ft. ↓

Deflection at quarter span due to slab weight

$$\Delta_{slab2} = \frac{57\, w_s\, L^4}{6144\, E_c\, I}$$

$$\Delta_{slab2} = \frac{57\left(0.80/12 \text{ in./ft.}\right)[(108.583)(12 \text{ in./ft.})]^4}{6{,}144(4{,}653.53)(260{,}403)}$$

$$= 1.471 \text{ in.} = 0.123 \text{ ft.} \downarrow$$

A.2.17.3 Deflections Due to Superimposed Dead Loads

Deflection due to barrier weight at midspan

$$\Delta_{barr1} = \frac{5\, w_{barr}\, L^4}{384\, E_c\, I_c}$$

where:

w_{barr} = Weight of the barrier = 0.109 kips/ft.

I_c = Moment of inertia of composite section = 651,886.0 in^4

$$\Delta_{barr1} = \frac{5\left(0.109/12 \text{ in./ft.}\right)[(108.583)(12 \text{ in./ft.})]^4}{384(4{,}653.53)(651{,}886.0)}$$

$$= 0.141 \text{ in.} = 0.0118 \text{ ft.} \downarrow$$

Deflection at quarter span due to barrier weight

$$\Delta_{barr2} = \frac{57\, w_{barr}\, L^4}{6144\, E_c\, I_c}$$

$$\Delta_{barr2} = \frac{57\left(0.109/12 \text{ in./ft.}\right)[(108.583)(12 \text{ in./ft.})]^4}{6{,}144(4{,}653.53)(651{,}886.0)}$$

$$= 0.08 \text{ in.} = 0.0067 \text{ ft.} \downarrow$$

Deflection due to wearing surface weight at midspan

$$\Delta_{ws1} = \frac{5\, w_{ws}\, L^4}{384\, E_c\, I_c}$$

where

w_{ws} = Weight of wearing surface = 0.128 kips/ft.

$$\Delta_{ws1} = \frac{5\left(0.128/12\text{ in./ft.}\right)[(108.583)(12\text{ in./ft.})]^4}{384(4,653.53)(651,886.0\ )}$$

$$= 0.132\text{ in.} = 0.011\text{ ft.} \downarrow$$

Deflection at quarter span due to wearing surface

$$\Delta_{ws2} = \frac{57\, w_{ws}\, L^4}{6144\, E_c\, I}$$

$$\Delta_{ws2} = \frac{57\left(0.128/12\text{ in./ft.}\right)[(108.583)(12\text{ in./ft.})]^4}{6,144(4,529.66)(657,658.4)}$$

$$= 0.094\text{ in.} = 0.0078\text{ ft.} \downarrow$$

A.2.17.4 Total Deflection Due to Dead Loads

The total deflection at midspan due to slab weight and superimposed loads is:

$$\Delta_{T1} = \Delta_{slab1} + \Delta_{barr1} + \Delta_{ws1}$$

$$= 0.172 + 0.0118 + 0.011 = 0.1948\text{ ft.} \downarrow$$

The total deflection at quarter span due to slab weight and superimposed loads is:

$$\Delta_{T2} = \Delta_{slab2} + \Delta_{barr2} + \Delta_{ws2}$$

$$= 0.123 + 0.0067 + 0.0078 = 0.1375\text{ ft.} \downarrow$$

The deflections due to live loads are not calculated in this example as they are not a design factor for TxDOT bridges.

Appendix A

Detailed Examples for Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design

DRAFT

August 29, 2005

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Table of Contents

A.1 Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design using AASHTO Standard Specifications

A.1.1 INTRODUCTION

Following is a detailed example showing sample calculations for the design of a typical interior AASHTO Type IV prestressed concrete girder supporting a single span bridge. The design is based on the *AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002* (AASHTO 2002). The guidelines provided by the TxDOT Bridge Design Manual (TxDOT 2001) are considered in the design. The number of strands and concrete strength at release and at service are optimized using the TxDOT methodology.

A.1.2 DESIGN PARAMETERS

The bridge considered for this design example has a span length of 110 ft. (c/c pier distance), a total width of 46 ft. and total roadway width of 44 ft. The bridge superstructure consists of six AASHTO Type IV girders spaced 8 ft. center-to-center, designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck. The wearing surface thickness is 1.5 in., which includes the thickness of any future wearing surface. T501 type rails are considered in the design. The design live load is taken as either HS 20 truck or HS 20 lane load, whichever produces larger effects. A relative humidity (RH) of 60% is considered in the design. The bridge cross-section is shown in Figure A.1.2.1.

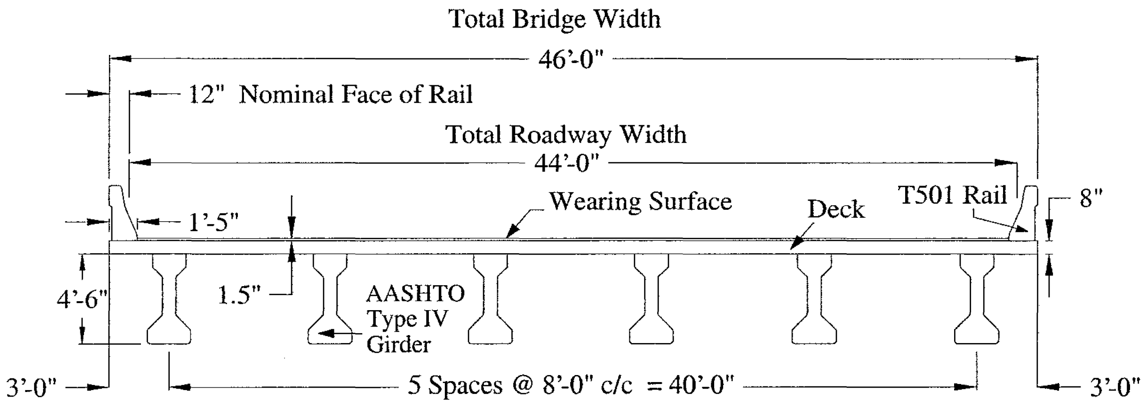

Figure A.1.2.1. Bridge Cross-Section Details

The design span and the overall girder length are based on the following calculations.

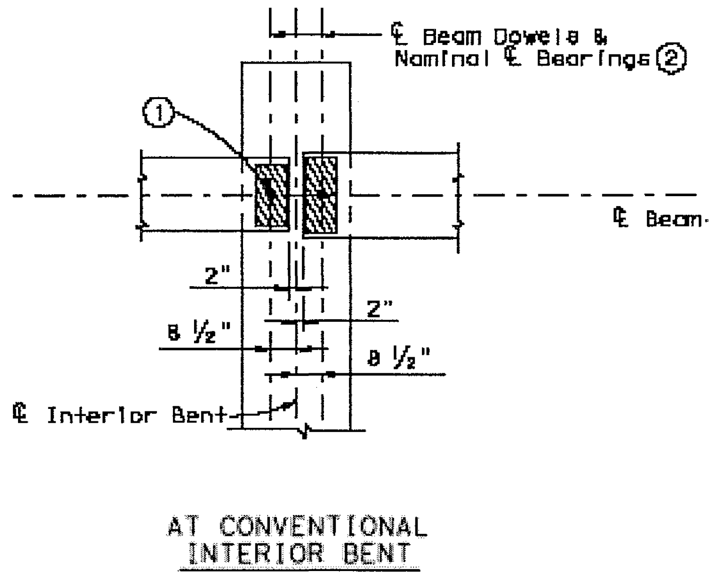

Figure A.1.2.2. Girder End Details
(TxDOT Standard Drawing 2001)

Span Length (c/c piers) = 110'-0"

From Figure A.1.2.2

Overall girder length = 110 ft. – 2(2 in.) = 109'-8"

Design Span = 110 ft. – 2(8.5 in.)
= 108'-7" = 108.583 ft. (c/c of bearing)

A.1.3 MATERIAL PROPERTIES

Cast in place (CIP) slab:

Thickness, t_s = 8.0 in.

Concrete Strength at 28-days, f'_c = 4,000 psi

Thickness of asphalt wearing surface (including any future wearing surface), t_w = 1.5 in.

Unit weight of concrete, w_c = 150 pcf

Precast girders: AASHTO Type IV

Concrete Strength at release, f'_{ci} = 4,000 psi (This value is taken as an initial estimate and will be finalized based on optimum design.)

Concrete Strength at 28 days, f'_c = 5,000 psi (This value is taken as initial estimate and will be finalized based on optimum design.)

Concrete unit weight, w_c = 150 pcf

Pretensioning Strands: ½ in. diameter, seven wire low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate stress, f'_s = 270,000 psi

Yield strength, f_y^* = 0.9 f'_s = 243,000 psi [STD Art. 9.1.2]

Initial pretensioning, f_{si} = 0.75 f'_s [STD Art. 9.15.1]
= 202,500 psi

Modulus of Elasticity, E_s = 28,000 ksi [STD Art. 9.16.2.1.2]

Nonprestressed reinforcement: Yield Strength, f_y = 60,000 psi

Unit weight of asphalt wearing surface = 140 pcf
[TxDOT recommendation]

T501 type barrier weight = 326 plf /side

A.1.4 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR GIRDER

A.1.4.1 Non-Composite Section

The section properties of an AASHTO Type IV girder as described in the TxDOT Bridge Design Manual (TxDOT 2001) are provided in Table A.1.4.1. The section geometry and strand pattern are shown in Figures A.1.4.1 and A.1.4.2, respectively.

Table A.1.4.1. Section Properties of AASHTO Type IV Girder [Notations as used in Figure A.1.4.1., Adapted from TxDOT Bridge Design Manual (TxDOT 2001)]

A	B	C	D	E	F	G	H	W	y_t	y_b	Area	I	Wt/lf
in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.^{2}	in.^{4}	lbs
20	26	8	54	9	23	6	8	8	29.25	24.75	788.4	260,403	821

where

I = Moment of inertia about the centroid of the non-composite precast girder, in.^{4}

y_b = Distance from centroid to the extreme bottom fiber of the non-composite precast girder, in.

y_t = Distance from centroid to the extreme top fiber of the non-composite precast girder, in.

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.^{3}
$= I/y_b = 260{,}403/24.75 = 10{,}521.33$ in.^{3}

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder, in.^{3}
$= I/y_t = 260{,}403/29.25 = 8{,}902.67$ in.^{3}

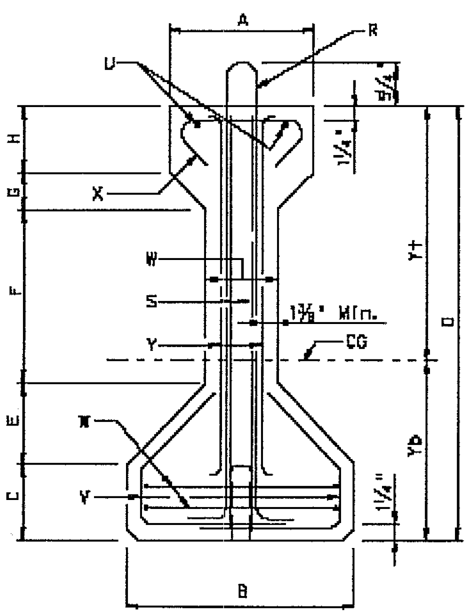

Figure A.1.4.1. Section Geometry of AASHTO Type IV Girder (TxDOT 2001)

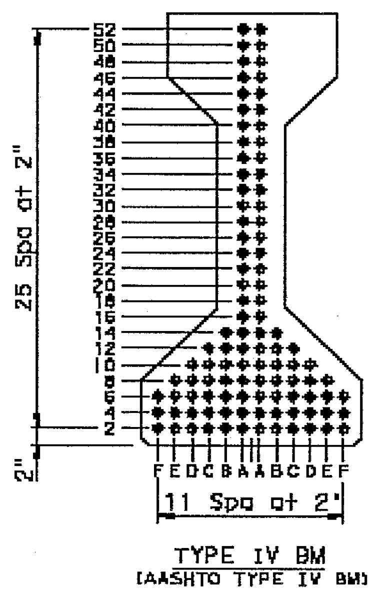

Figure A.1.4.2. Strand Pattern for AASHTO Type IV girder (TxDOT 2001)

A.1.4.2 Composite Section

A.1.4.2.1 Effective Web Width

[STD Art. 9.8.3]

Effective web width of the precast girder is lesser of:

[STD Art. 9.8.3.1]

b_e = 6*(flange thickness on either side of the web) + web + fillets
= 6(8 + 8) + 8 + 2(6) = 116 in.

or, b_e = Total top flange width = 20 in. (controls)

Effective web width, b_e = 20 in.

A.1.4.2.2 Effective Flange Width

The effective flange width is lesser of: [STD Art. 9.8.3.2]

¼ span length of girder: $\frac{108.583(12\text{ in./ft.})}{4} = 325.75$ in.

6*(effective slab thickness on each side of the effective web width) + effective web width: 6(8.0 + 8.0) + 20 = 116 in.

One-half the clear distance on each side of the effective web width + effective web width: For interior girders this is equivalent to the center-to-center distance between the adjacent girders.
8(12 in./ft.) + 20 in. = 96 in. (controls)

Effective flange width = 96 in.

A.1.4.2.3 Modular Ratio between Slab and Girder Concrete

Following the TxDOT Design Manual (TxDOT 2001) recommendation (Pg. 7-85), the modular ratio between the slab and the girder concrete is taken as 1. This assumption is used for service load design calculations. For flexural strength limit design, shear design, and deflection calculations, the actual modular ratio based on optimized concrete strengths is used.

$$n = \left(\frac{E_c \text{ for slab}}{E_c \text{ for girder}}\right) = 1$$

where n is the modular ratio between slab and girder concrete and E_c is the elastic modulus of concrete.

A.1.4.2.4 Transformed Section Properties

Transformed flange width = n*(effective flange width)
= (1)(96) = 96 in.

Transformed Flange Area = n*(effective flange width)(t_s)
= (1)(96) (8) = 768 in.^{2}

Table A.1.4.2. Properties of Composite Section

	Transformed Area A (in.^{2})	y_b in.	Ay_b in.^{3}	$A(y_{bc} - y_b)^2$	I in.^{4}	$I + A(y_{bc} - y_b)^2$ in.^{4}
Girder	788.4	24.75	19,512.9	212,231.53	260,403.0	472,634.5
Slab	768.0	58.00	44,544.0	217,868.93	4,096.0	221,964.9
$\sum$	1,556.4		64,056.9			694,599.5

A_c = Total area of composite section = 1,556.4 in.^{2}

h_c = Total height of composite section = 54 in. + 8 in. = 62 in.

I_c = Moment of inertia about the centroid of the composite section = 694,599.5 in^4

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.
= 64,056.9/1,556.4 = 41.157 in.

y_{tg} = Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.
= 54 - 41.157 = 12.843 in.

y_{tc} = Distance from the centroid of the composite section to extreme top fiber of the slab, in.
= 62 - 41.157 = 20.843 in.

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.^{3}
= I_c/y_{bc} = 694,599.5/41.157 = 16,876.83 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder, in.^{3}
= I_c/y_{tg} = 694,599.5/12.843 = 54,083.9 in.^{3}

S_{tc} = Section modulus of composite section referenced to the top fiber of the slab, in.^{3}
= I_c/y_{tc} = 694,599.5/20.843 = 33,325.31 in.^{3}

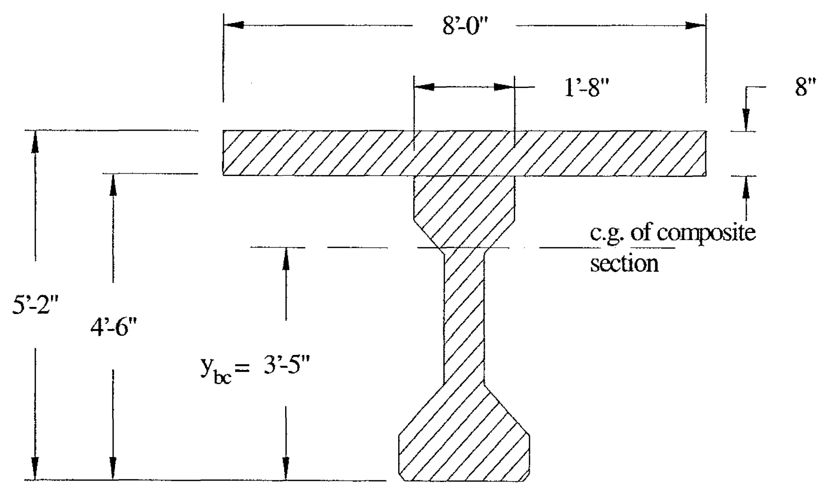

Figure A.1.4.3. Composite Section

A.1.5 SHEAR FORCES AND BENDING MOMENTS

The self-weight of the girder and the weight of slab act on the non-composite simple span structure, while the weight of the barriers, future wearing surface, and live load including impact load act on the composite simple span structure.

A.1.5.1 Shear Forces and Bending Moments due to Dead Loads

A.1.5.1.1 Dead Loads

Dead loads acting on the non-composite structure:

Self-weight of the girder = 0.821 kips/ft.
[TxDOT Bridge Design Manual (TxDOT 2001)]

Weight of cast-in-place (CIP) deck on each interior girder

$$= (0.150\text{ kcf})\left(\frac{8\text{ in.}}{12\text{ in./ft.}}\right)(8\text{ ft.}) = 0.800\text{ kips/ft.}$$

Total dead load on non-composite section
= 0.821 + 0.800 = 1.621 kips/ft.

A.1.5.1.2 Superimposed Dead Loads

The dead loads placed on the composite structure are distributed equally among all the girders.
[STD Art. 3.23.2.3.1.1 & TxDOT Bridge Design Manual Pg. 6-13]

Weight of T501 rails or barriers on each girder

$$= 2\left(\frac{326\text{ plf }/1000}{6\text{ girders}}\right) = 0.109\text{ kips/ft./girder}$$

Weight of 1.5 in. wearing surface

$$= (0.140\text{ kcf})\left(\frac{1.5\text{ in.}}{12\text{ in./ft.}}\right) = 0.0175\text{ ksf.}$$ This is applied over the entire clear roadway width of 44'-0".

Weight of wearing surface on each girder $= \dfrac{(0.0175\text{ ksf})(44.0\text{ ft.})}{6\text{ girders}}$

= 0.128 kips/ft./girder

Total superimposed dead load = 0.109 + 0.128 = 0.237 kips/ft.

A.1.5.1.3 Shear Forces and Bending Moments

Shear forces and bending moments for the girder due to dead loads, superimposed dead loads at every tenth of the design span, and at critical sections (hold down point or harp point and critical section for shear) are provided in this section. The bending moment (M) and shear force (V) due to uniform dead loads and uniform superimposed dead loads at any section at a distance x from the centerline of bearing are calculated using the following formulas, where the uniform dead load is denoted as w.

$$M = 0.5wx(L - x)$$

$$V = w(0.5L - x)$$

The critical section for shear is located at a distance $h_c/2$ from the face of the support. However, as the support dimensions are not specified in this study the critical section is measured from the centerline of bearing. This yields a conservative estimate of the design shear force.

Distance of critical section for shear from centerline of bearing
= 62/2 = 31 in. = 2.583 ft.

As per the recommendations of the TxDOT Bridge Design Manual (Chap. 7, Sec. 21), the distance of the hold down point (HD) from the centerline of bearing is taken as the lesser of:

(½ span length – span length/20) or (½ span length – 5 ft.)

$$\frac{108.583}{2} - \frac{108.583}{20} = 48.862 \text{ ft. or } \frac{108.583}{2} - 5 = 49.29 \text{ ft.}$$

The shear forces and bending moments due to dead loads and superimposed dead loads are shown in Table A.1.5.1.

Table A.1.5.1. Shear Forces and Bending Moments due to Dead and Superimposed Dead Loads

Distance from Bearing Centerline x	Section x/L	Dead Load				Superimposed Dead Loads		Total Dead Load	
		Girder Weight		Slab Weight					
		Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment
ft.		kips	k-ft.	kips	k-ft.	kips	k-ft.	kips	k-ft.
0.000	0.000	44.57	0.00	43.43	0.00	12.87	0.00	100.87	0.00
2.583	0.024 ($h_c/2$)	42.45	112.39	41.37	109.52	12.25	32.45	96.07	254.36
10.858	0.100	35.66	435.59	34.75	424.45	10.29	125.74	80.70	985.78
21.717	0.200	26.74	774.38	26.06	754.58	7.72	223.54	60.52	1,752.51
32.575	0.300	17.83	1,016.38	17.37	990.38	5.15	293.40	40.35	2,300.16
43.433	0.400	8.91	1,161.58	8.69	1,131.87	2.57	335.32	20.17	2,628.76
48.862	0.450 (HD)	4.46	1,197.87	4.34	1,167.24	1.29	345.79	10.09	2,710.90
54.292	0.500	0.00	1,209.98	0.00	1,179.03	0.00	349.29	0.00	2,738.29

A.1.5.2 Shear Forces and Bending Moments due to Live Load

A.1.5.2.1 Live Load

The AASHTO Standard Specifications require the live load to be taken as either HS 20 standard truck loading, lane loading or tandem loading; whichever yields the greatest moments and shears. For spans longer than 40 ft. tandem loading does not govern, thus only HS 20 truck loading and lane loading are investigated here.

[STD Art. 3.7.1.1]

The unfactored bending moments (M) and shear forces (V) due to HS 20 truck loading on a per-lane-basis are calculated using the following formulas given in the *PCI Design Manual* (PCI 2003).

Maximum bending moment due to HS 20 truck load

For $x/L = 0 - 0.333$

$$M = \frac{72(x)[(L-x)-9.33]}{L}$$

For $x/L = 0.333 - 0.5$

$$M = \frac{72(x)[(L-x)-4.67]}{L} - 112$$

Maximum shear force due to HS 20 truck load

For $x/L = 0 - 0.5$

$$V = \frac{72[(L-x)-9.33]}{L}$$

The bending moments and shear forces due to HS 20 lane load are calculated using the following formulas given in the *PCI Design Manual* (PCI 2003).

Maximum bending moment due to HS 20 lane load

$$M = \frac{P(x)(L-x)}{L} + 0.5(w)(x)(L-x)$$

Maximum shear force due to HS 20 lane load

$$V = \frac{Q(L-x)}{L} + (w)(\frac{L}{2} - x)$$

where

x = Distance from the centerline of bearing to the section at which bending moment or shear force is calculated, ft.

L = Design span length = 108.583 ft.

P = Concentrated load for moment = 18 kips

Q = Concentrated load for shear = 26 kips

w = Uniform load per linear foot of lane = 0.64 klf

Shear force and bending moment due to live load including impact loading is distributed to individual girders by multiplying the distribution factor and the impact factor as follows.

Bending moment due to live load including impact load
M_{LL+I} = (live load bending moment per lane) (DF) (1+I)

Shear force due to live load including impact load
V_{LL+I} = (live load shear force per lane) (DF) (1+I)

where DF is the live load distribution factor and I is the live load impact factor.

A.1.5.2.2 Live Load Distribution Factor for a Typical Interior Girder

The live load distribution factor for moment, for a precast prestressed concrete interior girder is given by the following expression

$$DF_{mom} = \frac{S}{5.5} = \frac{8.0}{5.5} = 1.4545 \text{ wheels/girder}$$ [STD Table 3.23.1]

where

S = Average spacing between girders in feet = 8 ft.

The live load distribution factor for individual girder is obtained as
$DF = DF_{mom}/2 = 0.727$ lanes/girder

For simplicity of calculation and because there is no significant difference, the distribution factor for moment is used also for shear as recommended by the TxDOT Bridge Design Manual (Chap. 6, Sec. 3, TxDOT 2001).

A.1.5.2.3 Live Load Impact

[STD Art. 3.8]

The live load impact factor is given by the following expression

$$I = \frac{50}{L + 125}$$ [STD Eq. 3-1]

where

I = Impact fraction to a maximum of 30%

L = Design span length in feet = 108.583 ft. [STD Art. 3.8.2.2]

$$I = \frac{50}{108.583 + 125} = 0.214$$

The impact factor for shear varies along the span according to the location of the truck, but the impact factor computed above is also used for shear for simplicity as recommended by the TxDOT Bridge Design Manual (TxDOT 2001).

Table A.1.5.2. Distributed Shear Forces and Bending Moments due to Live Load

Distance from Bearing Centerline x	Section x/L	HS 20 Truck Loading (controls)				HS 20 Lane Loading			
		Live Load		Live Load + Impact		Live Load		Live Load + Impact	
		Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment
ft.		kips	k-ft.	kips	k-ft.	kips	k-ft.	kips	k-ft.
0.000	0.000	65.81	0.00	58.11	0.00	60.75	0.00	53.64	0.00
2.583	0.024 ($h_c/2$)	64.10	165.57	56.60	146.19	58.47	133.00	51.63	117.44
10.858	0.100	58.61	636.44	51.75	561.95	51.20	515.46	45.20	455.13
21.717	0.200	51.41	1,116.52	45.40	985.84	41.65	916.38	36.77	809.12
32.575	0.300	44.21	1,440.25	39.04	1,271.67	32.10	1,202.75	28.34	1,061.97
43.433	0.400	37.01	1,629.82	32.68	1,439.05	22.55	1,374.57	19.91	1,213.68
48.862	0.450 (HD)	33.41	1,671.64	29.50	1,475.97	17.77	1,417.52	15.69	1,251.60
54.292	0.500	29.81	1,674.37	26.32	1,478.39	13.00	1,431.84	11.48	1,264.25

A.1.5.3 Load Combination

[STD Art. 3.22]

This design example considers only the dead and vehicular live loads. The wind load and the earthquake load are not included in the design, which is typical to the design of bridges in Texas. The general expression for group loading combinations for service load design (SLD) and load factor design (LFD) considering dead and live loads is given as:

$$\text{Group } (N) = \gamma^*[\beta_D{}^*D + \beta_L{}^*(L + I)]$$

where:

N = Group number

γ = Load factor given by STD Table 3.22.1.A.

β = Coefficient given by STD Table 3.22.1.A.

D = Dead load

L = Live load

I = Live load impact

Various group combinations provided by STD Table. 3.22.1.A are investigated and the following group combinations are found to be applicable in present case.

For service load design

Group I: This group combination is used for design of members for 100% basic unit stress. [STD Table 3.22.1A]

γ = 1.0

β_D = 1.0

β_L = 1.0

Group (I) = 1.0*D + 1.0*(L+I)

For load factor design

Group I: This load combination is the general load combination for load factor design relating to the normal vehicular use of the bridge. [STD Table 3.22.1A]

γ = 1.3

β_D = 1.0 for flexural and tension members.

β_L = 1.67

Group (I) = 1.3[1.0*D + 1.67*(L+I)]

A.1.6 ESTIMATION OF REQUIRED PRESTRESS

A.1.6.1 Service Load Stresses at Midspan

The required number of strands is usually governed by concrete tensile stress at the bottom fiber of the girder at midspan section. The service load combination, Group I is used to evaluate the bottom fiber stresses at the midspan section. The calculation for compressive stress in the top fiber of the girder at midspan section under Group I service load combination is also shown in the following section.

Tensile stress at bottom fiber of the girder at midspan due to applied loads

$$f_b = \frac{M_g + M_S}{S_b} + \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

Compressive stress at top fiber of the girder at midspan due to applied loads

$$f_t = \frac{M_g + M_S}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}}$$

where:

f_b = Concrete stress at the bottom fiber of the girder at the midspan section, ksi

f_t = Concrete stress at the top fiber of the girder at the midspan section, ksi

M_g = Moment due to girder self-weight at the midspan section of the girder = 1,209.98 k-ft.

M_S = Moment due to slab weight at the midspan section of the girder = 1,179.03 k-ft.

M_{SDL} = Moment due to superimposed dead loads at the midspan section of the girder = 349.29 k-ft.

M_{LL+I} = Moment due to live load including impact load at the midspan section of the girder = 1,478.39 k-ft.

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

S_t = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8,902.67 in.^{3}

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder = 16,876.83 in.^{3}

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.^{3}

Substituting the bending moments and section modulus values, the stresses at bottom fiber (f_b) and top fiber (f_t) of the girder at the midspan section are:

$$f_b = \frac{(1{,}209.98 + 1{,}179.03)(12 \text{ in./ft.})}{10{,}521.33} + \frac{(349.29 + 1{,}478.39)(12 \text{ in./ft.})}{16{,}876.83}$$
$$= 4.024 \text{ ksi}$$

$$f_t = \frac{(1{,}209.98 + 1{,}179.03)(12 \text{ in./ft.})}{8{,}902.67} + \frac{(349.29 + 1{,}478.39)(12 \text{ in./ft.})}{54{,}083.9}$$
$$= 3.626 \text{ ksi}$$

The stresses at the top and bottom fibers of the girder at the hold down point, midspan and top fiber of the slab are calculated in a similar fashion as shown above and summarized in Table A.1.6.1.

Table A.1.6.1. Summary of Stresses due to Applied Loads

Load	Stresses in Girder				Stresses in Slab at Midspan
	Stress at Hold Down (*HD*)		Stress at Midspan		
	Top Fiber (psi)	Bottom Fiber (psi)	Top Fiber (psi)	Bottom Fiber (psi)	Top Fiber (psi)
Girder Self-weight	1,614.63	-1,366.22	1,630.94	-1,380.03	-
Slab Weight	1,573.33	-1,331.28	1,589.22	-1,344.73	-
Superimposed Dead Load	76.72	-245.87	77.50	-248.35	125.77
Total Dead Load	3,264.68	-2,943.37	3,297.66	-2,973.10	125.77
Live Load	327.49	-1,049.47	328.02	-1,051.19	532.35
Total Load	3,592.17	-3,992.84	3,625.68	-4,024.29	658.12

(Negative values indicate tensile stresses)

A.1.6.2 Allowable Stress Limit

At service load conditions, the allowable tensile stress for members with bonded prestressed reinforcement is

$$F_b = 6\sqrt{f_c'} = 6\sqrt{5{,}000}\left(\frac{1}{1{,}000}\right) = 0.4242 \text{ ksi} \qquad \text{[STD Art. 9.15.2.2]}$$

A.1.6.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – allowable tensile stress at final = $f_b - F_b$

$f_{b\text{-reqd.}} = 4.024 - 0.4242 = 3.60$ ksi

Assuming the eccentricity of the prestressing strands at midspan (e_c) as the distance from the centroid of the girder to the bottom fiber of the girder (PSTRS 14 methodology, TxDOT 2001)

$e_c = y_b = 24.75$ in.

Bottom fiber stress due to prestress after losses:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b}$$

where:

P_{se} = Effective pretension force after all losses, kips

A = Area of girder cross-section = 788.4 in.^{2}

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

Required pretension is calculated by substituting the corresponding values in above equation as follows:

$$3.60 = \frac{P_{se}}{788.4} + \frac{P_{se}(24.75)}{10{,}521.33}$$

Solving for P_{se},
P_{se} = 994.27 kips

Assuming final losses = 20% of initial prestress, f_{si} (TxDOT 2001)

Assumed final losses = 0.2(202.5) = 40.5 ksi

The prestress force per strand after losses
= (cross-sectional area of one strand) [f_{si} – losses]
= 0.153(202.5 – 40.5] = 24.78 kips

Number of prestressing strands required = 994.27/24.78 = 40.12

Try 42 – ½ in. diameter, 270 ksi low-relaxation strands as an initial estimate.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 6(8)}{42} = 20.18 \text{ in.}$$

Available prestressing force
P_{se} = 42(24.78) = 1040.76 kips

Stress at bottom fiber of the girder at midspan due to prestressing, after losses

$$f_b = \frac{1{,}040.76}{788.4} + \frac{1{,}040.76\,(20.18)}{10{,}521.33}$$

$$= 1.320 + 1.996 = 3.316 \text{ ksi} < f_{b\text{-}reqd.} = 3.60 \text{ ksi}$$

Try 44 – ½ in. diameter, 270 ksi low-relaxation strands as an initial estimate.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 8(8)}{44} = 20.02 \text{ in.}$$

Available prestressing force
P_{se} = 44(24.78) = 1,090.32 kips

Stress at bottom fiber of the girder at midspan due to prestressing, after losses

$$f_b = \frac{1{,}090.32}{788.4} + \frac{1{,}090.32\ (20.02)}{10{,}521.33}$$

$= 1.383 + 2.074 = 3.457$ ksi $< f_{b\text{-}reqd.} = 3.60$ ksi

Try 46 – ½ in. diameter, 270 ksi low-relaxation strands as an initial estimate

Effective strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2{+}4{+}6) + 10(8)}{46} = 19.88 \text{ in.}$$

Available prestressing force is

$P_{se} = 46(24.78) = 1{,}139.88$ kips

Stress at bottom fiber of the girder at midspan due to prestressing, after losses

$$f_b = \frac{1{,}139.88}{788.4} + \frac{1{,}139.88\ (19.88)}{10{,}521.33}$$

$= 1.446 + 2.153 = 3.599$ ksi $\sim f_{b\text{-}reqd.} = 3.601$ ksi

Therefore 46 strands are used as a preliminary estimate for the number of strands. The strand arrangement is shown in Figure A.1.6.1.

Number of Strands	Distance from bottom (in.)
10	8
12	6
12	4
12	2

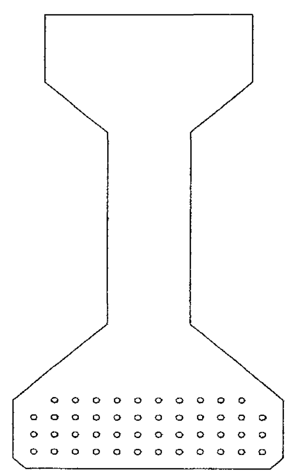

Figure A.1.6.1. Initial Strand Arrangement

The distance from the centroid of the strands to the bottom fiber of the girder (y_{bs}) is calculated as:

$y_{bs} = y_b - e_c = 24.75 - 19.88 = 4.87$ in.

A.1.7 PRESTRESS LOSSES

[STD Art. 9.16.2]

Total prestress losses = $SH + ES + CR_C + CR_S$ [STD Eq. 9-3]

where:

SH = Loss of prestress due to concrete shrinkage, ksi

ES = Loss of prestress due to elastic shortening, ksi

CR_C = Loss of prestress due to creep of concrete, ksi

CR_S = Loss of prestress due to relaxation of pretensioning steel, ksi

Number of strands = 46

A number of iterations based on TxDOT methodology (TxDOT 2001) will be performed to arrive at the optimum number of strands, required concrete strength at release (f'_{ci}) and required concrete strength at service (f'_c).

A.1.7.1 Iteration 1

A.1.7.1.1 ɘ Shrinkage

[STD Art. 9.16.2.1.1]

For pretensioned members, the loss in prestress due to concrete shrinkage is given as

$$SH = 17{,}000 - 150\,RH \quad \text{[STD Eq. 9-4]}$$

where:

RH is the relative humidity = 60%

$$SH = [17{,}000 - 150(60)]\frac{1}{1{,}000} = 8.0 \text{ ksi}$$

A.1.7.1.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

For pretensioned members, the loss in prestress due to elastic shortening is given as

$$ES = \frac{E_s}{E_{ci}} f_{cir} \quad \text{[STD Eq. 9-6]}$$

where:

f_{cir} = Average concrete stress at the center of gravity of the pretensioning steel due to the pretensioning force and the dead load of girder immediately after transfer, ksi

$$= \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

P_{si} = Pretension force after allowing for the initial losses, kips

As the initial losses are unknown at this point, 8% initial loss in prestress is assumed as a first estimate

P_{si} = (number of strands)(area of each strand)[0.92(0.75 f'_s)]
= 46(0.153)(0.92)(0.75)(270) = 1,311.18 kips

M_g = Moment due to girder self-weight at midspan, k-ft.
= 1,209.98 k-ft.

e_c = Eccentricity of the prestressing strands at the midspan
= 19.88 in.

$$f_{cir} = \frac{1{,}311.18}{788.4} + \frac{1{,}311.18(19.88)^2}{260{,}403} - \frac{1{,}209.98(12\text{ in./ft.})(19.88)}{260{,}403}$$

= 1.663 + 1.990 – 1.108 = 2.545 ksi

Initial estimate for concrete strength at release, f'_{ci} = 4,000 psi

Modulus of elasticity of girder concrete at release is given as

$E_{ci} = 33(w_c)^{3/2}\sqrt{f'_{ci}}$ [STD Eq. 9-8]

$$= [33(150)^{3/2}\sqrt{4{,}000}]\left(\frac{1}{1{,}000}\right) = 3{,}834.25 \text{ ksi}$$

Modulus of elasticity of prestressing steel, E_s = 28,000 ksi

Prestress loss due to elastic shortening is

$$ES = \left[\frac{28{,}000}{3{,}834.25}\right](2.545) = 18.59 \text{ ksi}$$

A.1.7.1.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

The loss in prestress due to the creep of concrete is specified to be calculated using the following formula

$CR_C = 12f_{cir} - 7f_{cds}$ [STD Eq. 9-9]

where:

f_{cds} = Concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied, ksi

$$= \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_{SDL} = Moment due to superimposed dead load at midspan section = 349.29 k-ft.

M_S = Moment due to slab weight at midspan section = 1,179.03 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder = 24.75 – 19.88 = 4.87 in.

I = Moment of inertia of the non-composite section = 260,403 in.^{4}

I_c = Moment of inertia of composite section = 694,599.5 in.^{4}

$$f_{cds} = \frac{1{,}179.03(12\text{ in./ft.})(19.88)}{260{,}403} + \frac{(349.29)(12\text{ in./ft.})(41.157 - 4.87)}{694{,}599.5}$$

$$= 1.080 + 0.219 = 1.299 \text{ ksi}$$

Prestress loss due to creep of concrete is
CR_C = 12(2.545) – 7(1.299) = 21.45 ksi

A.1.7.1.4 Relaxation of Prestressing Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strands, the prestress loss due to relaxation of prestressing steel is calculated using the following formula.

$$CR_S = 5{,}000 - 0.10ES - 0.05(SH + CR_C) \quad \text{[STD Eq. 9-10A]}$$

where the variables are same as defined in Section A.1.7 expressed in psi units.

$$CR_S = [5{,}000 - 0.10(18{,}590) - 0.05(8{,}000 + 21{,}450)]\left(\frac{1}{1{,}000}\right)$$

$$= 1.669 \text{ ksi}$$

The *PCI Design Manual* (PCI 2003) considers only the elastic shortening loss in the calculation of total initial prestress loss whereas, the TxDOT Bridge Design Manual (Pg. 7-85, TxDOT 2001) recommends that 50% of the final steel relaxation loss shall also be considered for calculation of total initial prestress loss given as:
[elastic shortening loss + 0.50*(total steel relaxation loss)]

Using the TxDOT Bridge Design Manual (TxDOT 2001) recommendations, the initial prestress loss is calculated as follows.

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75f_s'}$$

$$= \frac{[18.59 + 0.5(1.669)]100}{0.75(270)} = 9.59\% > 8\% \text{ (assumed value of}$$

initial prestress loss)

Therefore, another trial is required assuming 9.59% initial prestress loss.

The change in initial prestress loss will not affect the prestress loss due to concrete shrinkage. Therefore, the next trials will involve updating the losses due to elastic shortening, steel relaxation and creep of concrete.

Based on the initial prestress loss value of 9.59%, the pretension force after allowing for the initial losses is calculated as follows.

P_{si} = (number of strands)(area of each strand)[0.904(0.75 f_s')]
= 46(0.153)(0.904)(0.75)(270) = 1,288.38 kips

Loss in prestress due to elastic shortening

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$f_{cir} = \frac{1{,}288.38}{788.4} + \frac{1{,}288.38(19.88)^2}{260{,}403} - \frac{1{,}209.98(12 \text{ in./ft.})(19.88)}{260{,}403}$$

$$= 1.634 + 1.955 - 1.108 = 2.481 \text{ ksi}$$

E_s = 28,000 ksi

E_{ci} = 3,834.25 ksi

$$ES = \left[\frac{28{,}000}{3{,}834.25}\right](2.481) = 18.12 \text{ ksi}$$

Loss in prestress due to creep of concrete

$CR_C = 12f_{cir} - 7f_{cds}$

The value of f_{cds} is independent of the initial prestressing force value and will be same as calculated in Section A.1.7.1.3.
f_{cds} = 1.299 ksi

$CR_C = 12(2.481) - 7(1.299) = 20.68$ ksi

Loss in prestress due to relaxation of steel

$$CR_S = 5{,}000 - 0.10\,ES - 0.05(SH + CR_C)$$

$$= [5{,}000 - 0.10(18{,}120) - 0.05(8{,}000 + 20{,}680)]\left(\frac{1}{1000}\right)$$

$$= 1.754 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75f_s'}$$

$$= \frac{[18.12 + 0.5(1.754)]100}{0.75(270)} = 9.38\% < 9.59\%$$ (assumed value for initial prestress loss)

Therefore, another trial is required assuming 9.38% initial prestress loss.

Based on the initial prestress loss value of 9.38%, the pretension force after allowing for the initial losses is calculated as follows.

P_{si} = (number of strands)(area of each strand)[0.906 (0.75 f_s')]
= 46(0.153)(0.906)(0.75)(270) = 1,291.23 kips

Loss in prestress due to elastic shortening

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$f_{cir} = \frac{1{,}291.23}{788.4} + \frac{1{,}291.23(19.88)^2}{260{,}403} - \frac{1{,}209.98(12\text{ in./ft.})(19.88)}{260{,}403}$$

$$= 1.638 + 1.960 - 1.108 = 2.490 \text{ ksi}$$

E_s = 28,000 ksi

E_{ci} = 3,834.25 ksi

$$ES = \left[\frac{28{,}000}{3{,}834.25}\right](2.490) = 18.18 \text{ ksi}$$

Loss in prestress due to creep of concrete

$CR_C = 12f_{cir} - 7f_{cds}$

$f_{cds} = 1.299$ ksi

$CR_C = 12(2.490) - 7(1.299) = 20.79$ ksi.

Loss in prestress due to relaxation of steel

$CR_S = 5{,}000 - 0.10\ ES - 0.05(SH + CR_C)$

$= [5{,}000 - 0.10(18{,}180) - 0.05(8{,}000 + 20{,}790)]\left(\dfrac{1}{1{,}000}\right)$

$= 1.743$ ksi

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75f_s'}$$

$= \dfrac{[18.18 + 0.5(1.743)]100}{0.75(270)} = 9.41\% \approx 9.38\%$ (assumed value of initial prestress loss)

A.1.7.1.5 Total Losses at Transfer

Total prestress loss at transfer $= (ES + \frac{1}{2}CR_s)$

$= [18.18 + 0.5(1.743)] = 19.05$ ksi

Effective initial prestress, $f_{si} = 202.5 - 19.05 = 183.45$ ksi

P_{si} = Effective pretension after allowing for the initial prestress loss

= (number of strands)(area of strand)(f_{si})

$= 46(0.153)(183.45) = 1{,}291.12$ kips

A.1.7.1.6 Total Losses at Service

Loss in prestress due to concrete shrinkage, $SH = 8.0$ ksi

Loss in prestress due to elastic shortening, $ES = 18.18$ ksi

Loss in prestress due to creep of concrete, $CR_C = 20.79$ ksi

Loss in prestress due to steel relaxation, $CR_S = 1.743$ ksi

Total final loss in prestress $= SH + ES + CR_C + CR_S$

$= 8.0 + 18.18 + 20.79 + 1.743 = 48.71$ ksi

or, $\dfrac{48.71(100)}{0.75(270)} = 24.06$ %

Effective final prestress, $f_{se} = 0.75(270) - 48.71 = 153.79$ ksi

P_{se} = Effective pretension after allowing for the final prestress loss

= (number of strands)(area of strand)(effective final prestress)

= 46(0.153)(153.79) = 1,082.37 kips

A.1.7.1.7 Final Stresses at Midspan

The number of strands is updated based on the final stress at the bottom fiber of the girder at the midspan section.

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress, f_{bf}, is calculated as follows.

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} = \frac{1{,}082.37}{788.4} + \frac{1{,}082.37\,(19.88)}{10{,}521.33}$$

$= 1.373 + 2.045 = 3.418$ ksi $< f_{b\text{-}reqd.} = 3.600$ ksi (N.G)

($f_{b\text{-}reqd.}$ calculations are presented in Section A.1.6.3)

Try 48 – ½ in. diameter, 270 ksi low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8) + 2(10)}{48} = 19.67 \text{ in.}$$

Effective pretension after allowing for the final prestress loss
P_{se} = 48(0.153)(153.79) = 1,129.43 kips

Final stress at the bottom fiber of the girder at midspan section due to effective prestress

$$f_{bf} = \frac{1{,}129.43}{788.4} + \frac{1{,}129.43\,(19.67)}{10{,}521.33}$$

$= 1.432 + 2.11 = 3.542$ ksi $< f_{b\text{-}reqd.} = 3.600$ ksi (N.G.)

Try 50 – ½ in. diameter, 270 ksi low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8) + 4(10)}{50} = 19.47 \text{ in.}$$

Effective pretension after allowing for the final prestress loss
P_{se} = 50(0.153)(153.79) = 1,176.49 kips

Final stress at the bottom fiber of the girder at midspan section due to effective prestress

$$f_{bf} = \frac{1,176.49}{788.4} + \frac{1,176.49\,(19.47)}{10,521.33}$$

$$= 1.492 + 2.177 = 3.669 \text{ ksi} > f_{b\text{-}reqd.} = 3.600 \text{ ksi} \qquad \text{(O.K.)}$$

Therefore use 50 – ½ in. diameter, 270 ksi low-relaxation strands.

Concrete stress at the top fiber of the girder due to effective prestress and applied loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1,176.49}{788.4} - \frac{1,176.49\,(19.47)}{8,902.67} + 3.626$$

$$= 1.492 - 2.573 + 3.626 = 2.545 \text{ ksi}$$

(f_t calculations are presented in Section A.1.6.1)

A.1.7.1.8 Initial Stresses at Hold Down Point

The concrete strength at release, f'_{ci}, is updated based on the initial stress at the bottom fiber of the girder at the hold down point.

Prestressing force after allowing for initial prestress loss

P_{si} = (number of strands)(area of strand)(effective initial prestress)

= 50(0.153)(183.45) = 1,403.39 kips

(Effective initial prestress calculations are presented in Section A.1.7.1.5.)

Initial concrete stress at top fiber of the girder at the hold down point due to self-weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\,e_c}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at hold down point based on overall girder length of 109'-8".
= $0.5wx(L - x)$

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Distance of hold down point from the end of the girder = HD + (distance from centerline of bearing to the girder end)

HD = Hold down point distance from centerline of the bearing = 48.862 ft. (see Sec. A.1.5.1.3)

x = 48.862 + 0.542 = 49.404 ft.

M_g = 0.5(0.821)(49.404)(109.67 - 49.404) = 1,222.22 k-ft.

$$f_{ti} = \frac{1{,}403.39}{788.4} - \frac{1{,}403.39\,(19.47)}{8{,}902.67} + \frac{1{,}222.22(12\text{ in./ft.})}{8{,}902.67}$$

$$= 1.78 - 3.069 + 1.647 = 0.358 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of the girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1{,}403.39}{788.4} + \frac{1{,}403.39\,(19.47)}{10{,}521.33} - \frac{1{,}222.22(12\text{ in./ft.})}{10{,}521.33}$$

$$= 1.78 + 2.597 - 1.394 = 2.983 \text{ ksi}$$

Compression stress limit for pretensioned members at transfer stage is $0.6 f'_{ci}$ [STD Art. 9.15.2.1]

Therefore, $f'_{ci\text{-}reqd.} = \dfrac{2983}{0.6} = 4{,}971.67$ psi

A.1.7.2 Iteration 2

A second iteration is carried out to determine the prestress losses and subsequently estimate the required concrete strength at release and at service using the following parameters determined in the previous iteration.

Number of strands = 50
Concrete Strength at release, f'_{ci} = 4971.67 psi

A.1.7.2.1 Concrete Shrinkage

[STD Art. 9.16.2.1.1]

For pretensioned members, the loss in prestress due to concrete shrinkage is given as

$$SH = 17{,}000 - 150\,RH \quad \text{[STD Eq. 9-4]}$$

where RH is the relative humidity = 60%

$$SH = [17{,}000 - 150(60)]\frac{1}{1{,}000} = 8.0 \text{ ksi}$$

A.1.7.2.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

For pretensioned members, the loss in prestress due to elastic shortening is given as

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where:

f_{cir} = Average concrete stress at the center of gravity of the pretensioning steel due to the pretensioning force and the dead load of girder immediately after transfer, ksi

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

P_{si} = Pretension force after allowing for the initial losses, kips

As the initial losses are dependent on the elastic shortening and steel relaxation loss which are yet to be determined, the initial loss value of 9.41% obtained in the last trial of iteration 1 is taken as an initial estimate for initial loss in prestress.

P_{si} = (number of strands)(area of strand)[0.9059(0.75 f_s')]
= 50(0.153)(0.9059)(0.75)(270) = 1,403.35 kips

M_g = Moment due to girder self-weight at midspan, k-ft.
= 1,209.98 k-ft.

e_c = Eccentricity of the prestressing strands at the midspan
= 19.47 in.

$$f_{cir} = \frac{1{,}403.35}{788.4} + \frac{1{,}403.35(19.47)^2}{260{,}403} - \frac{1{,}209.98(12\text{ in./ft.})(19.47)}{260{,}403}$$

= 1.78 + 2.043 – 1.086 = 2.737 ksi

Concrete strength at release, f'_{ci} = 4,971.67 psi

Modulus of elasticity of girder concrete at release is given as

$$E_{ci} = 33(w_c)^{3/2}\sqrt{f'_{ci}}$$ [STD Eq. 9-8]

$$= [33(150)^{3/2}\sqrt{4{,}971.67}\,]\left(\frac{1}{1{,}000}\right) = 4{,}274.66 \text{ ksi}$$

Modulus of elasticity of prestressing steel, E_s = 28,000 ksi

Prestress loss due to elastic shortening is

$$ES = \left[\frac{28{,}000}{4{,}274.66}\right](2.737) = 17.93 \text{ ksi}$$

A.1.7.2.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

The loss in prestress due to creep of concrete is specified to be calculated using the following formula.

$$CR_C = 12f_{cir} - 7f_{cds} \qquad \text{[STD Eq. 9-9]}$$

where:

$$f_{cds} = \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_{SDL} = Moment due to superimposed dead load at midspan section = 349.29 k-ft.

M_S = Moment due to slab weight at midspan section = 1,179.03 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder = 24.75 – 19.47 = 5.28 in.

I = Moment of inertia of the non-composite section = 260,403 in.^{4}

I_c = Moment of inertia of composite section = 694599.5 in.^{4}

$$f_{cds} = \frac{1{,}179.03(12 \text{ in./ft.})(19.47)}{260{,}403} + \frac{(349.29)(12 \text{ in./ft.})(41.157 - 5.28)}{694{,}599.5}$$

$$= 1.058 + 0.216 = 1.274 \text{ ksi}$$

Prestress loss due to creep of concrete is

$$CR_C = 12(2.737) - 7(1.274) = 23.93 \text{ ksi}$$

A.1.7.2.4 Relaxation of Pretensioning Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strands, prestress loss due to relaxation of prestressing steel is calculated using the following formula.

$$CR_S = 5{,}000 - 0.10\ ES - 0.05(SH + CR_C) \qquad \text{[STD Eq. 9-10A]}$$

$$CR_S = [5{,}000 - 0.10(17{,}930) - 0.05(8{,}000{+}23{,}930)]\left(\frac{1}{1000}\right)$$

$$= 1.61 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75 f_s'}$$

$$= \frac{[17.93 + 0.5(1.61)]100}{0.75(270)} = 9.25\% < 9.41\%$$ (assumed value of initial prestress loss)

Therefore another trial is required assuming 9.25% initial prestress loss.

The change in initial prestress loss will not affect the prestress loss due to concrete shrinkage. Therefore, the next trial will involve updating the losses due to elastic shortening, steel relaxation, and creep of concrete.

Based on the initial prestress loss value of 9.25%, the pretension force after allowing for the initial losses is calculated as follows

$$P_{si} = \text{(number of strands)(area of each strand)}[0.9075(0.75\, f_s')]$$

$$= 50(0.153)(0.9075)(0.75)(270) = 1{,}405.83 \text{ kips}$$

Loss in prestress due to elastic shortening

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$= \frac{1{,}405.83}{788.4} + \frac{1{,}405.83(19.47)^2}{260{,}403} - \frac{1{,}209.98(12 \text{ in./ft.})(19.47)}{260{,}403}$$

$$= 1.783 + 2.046 - 1.086 = 2.743 \text{ ksi}$$

$$E_s = 28{,}000 \text{ ksi}$$

$$E_{ci} = 4{,}274.66 \text{ ksi}$$

Prestress loss due to elastic shortening is

$$ES = \left[\frac{28{,}000}{4{,}274.66}\right](2.743) = 17.97 \text{ ksi}$$

Loss in prestress due to creep of concrete
$CR_C = 12f_{cir} - 7f_{cds}$

The value of f_{cds} is independent of the initial prestressing force value and will be the same as calculated in Section A.1.7.2.3.
$f_{cds} = 1.274$ ksi

$CR_C = 12(2.743) - 7(1.274) = 24.0$ ksi

Loss in prestress due to relaxation of steel
$CR_S = 5{,}000 - 0.10\ ES - 0.05(SH + CR_C)$

$$= [5{,}000 - 0.10(17{,}970) - 0.05(8{,}000 + 24{,}000)]\left(\frac{1}{1000}\right)$$

$= 1.603$ ksi

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75f_s'}$$

$$= \frac{[17.97 + 0.5(1.603)]100}{0.75(270)} = 9.27\% \approx 9.25\% \text{ (assumed value}$$
for initial prestress loss)

A.1.7.2.5 Total Losses at Transfer

$$\text{Total prestress loss at transfer} = (ES + \frac{1}{2}CR_s)$$

$= [17.97 + 0.5(1.603)] = 18.77$ ksi

Effective initial prestress, $f_{si} = 202.5 - 18.77 = 183.73$ ksi

P_{si} = Effective pretension after allowing for the initial prestress loss

= (number of strands)(area of strand)(f_{si})

= 50(0.153)(183.73) = 1,405.53 kips

A.1.7.2.6 Total Losses at Service

Loss in prestress due to concrete shrinkage, $SH = 8.0$ ksi

Loss in prestress due to elastic shortening, $ES = 17.97$ ksi

Loss in prestress due to creep of concrete, $CR_C = 24.0$ ksi

Loss in prestress due to steel relaxation, $CR_S = 1.603$ ksi

Total final loss in prestress = $SH + ES + CR_C + CR_S$

$= 8.0 + 17.97 + 24.0 + 1.603 = 51.57$ ksi

or $\dfrac{51.57(100)}{0.75(270)} = 25.47$ %

Effective final prestress, $f_{se} = 0.75(270) - 51.57 = 150.93$ ksi

P_{se} = Effective pretension after allowing for the final prestress loss

= (number of strands)(area of strand)(effective final prestress)

= 50(0.153)(150.93) = 1,154.61 kips

A.1.7.2.7 Final Stresses at Midspan

Concrete stress at top fiber of the girder at the midspan section due to applied loads and effective prestress

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1{,}154.61}{788.4} - \frac{1{,}154.61\ (19.47)}{8{,}902.67} + 3.626$$

$$= 1.464 - 2.525 + 3.626 = 2.565 \text{ ksi}$$

(f_t calculations are presented in Section A.1.6.1)

Compressive stress limit under service load combination is $0.6\,f'_c$

[STD Art. 9.15.2.2]

$$f'_c\text{-}reqd. = \frac{2{,}565}{0.60} = 4{,}275 \text{ psi}$$

Concrete stress at top fiber of the girder at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

$$= \frac{1{,}154.61}{788.4} - \frac{1{,}154.61\ (19.47)}{8{,}902.67} + \frac{(1{,}209.98 + 1{,}179.03)(12 \text{ in./ft.})}{8{,}902.67} + \frac{349.29(12 \text{ in./ft.})}{54{,}083.9}$$

$$= 1.464 - 2.525 + 3.22 + 0.077 = 2.236 \text{ ksi}$$

Compressive stress limit for effective prestress + permanent dead loads = $0.4\,f'_c$ [STD Art. 9.15.2.2]

$$f'_c\text{-}reqd. = \frac{2{,}236}{0.40} = 5{,}590 \text{ psi} \quad \text{(controls)}$$

Concrete stress at top fiber of the girder at midspan due to live load + ½(effective prestress + dead loads)

$$f_{tf} = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}\right)$$

$$= \frac{1{,}478.39(12\text{ in./ft.})}{54083.9} + 0.5\left\{\frac{1{,}154.61}{788.4} - \frac{1{,}154.61(19.47)}{8{,}902.67} + \frac{(1{,}209.98 + 1{,}179.03)(12\text{ in./ft.})}{8{,}902.67} + \frac{349.29(12\text{ in./ft.})}{54{,}083.9}\right\}$$

$$= 0.328 + 0.5(1.464 - 2.525 + 3.22 + 0.077) = 1.446\text{ ksi}$$

Allowable limit for compressive stress due to live load + ½(effective prestress + dead loads) = 0.4 f'_c [STD Art. 9.15.2.2]

$$f'_c\text{-}reqd. = \frac{1{,}446}{0.40} = 3{,}615\text{ psi}$$

Tensile stress at the bottom fiber of the girder at midspan due to service loads

$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - f_b$ (f_b calculations are presented in Sec. A.1.6.1)

$$= \frac{1{,}154.61}{788.4} + \frac{1{,}154.61(19.47)}{10{,}521.33} - 4.024$$

= 1.464 + 2.14 – 4.024 = – 0.420 ksi (negative sign indicates tensile stress)

For members with bonded reinforcement allowable tension in the precompressed tensile zone = $6\sqrt{f'_c}$ [STD Art. 9.15.2.2]

$$f'_c\text{-}reqd. = \left(\frac{420}{6}\right)^2 = 4{,}900\text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations. The required concrete strength at service is determined to be 5,590 psi.

A.1.7.2.8 Initial Stresses at Hold Down Point

Prestressing force after allowing for initial prestress loss

P_{si} = (number of strands)(area of strand)(effective initial prestress)

= 50(0.153)(183.73) = 1,405.53 kips

(Effective initial prestress calculations are presented in Section A.1.7.2.5)

Initial concrete stress at top fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\,e_c}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at the hold down point based on overall girder length of 109'-8"
= 1,222.22 k-ft. (see Section A.1.7.1.8)

$$f_{ti} = \frac{1{,}405.53}{788.4} - \frac{1{,}405.53(19.47)}{8{,}902.67} + \frac{1{,}222.22(12\text{ in./ft.})}{8{,}902.67}$$

= 1.783 – 3.074 + 1.647 = 0.356 ksi

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1{,}405.53}{788.4} + \frac{1{,}405.53(19.47)}{10{,}521.33} - \frac{1{,}222.22(12\text{ in./ft.})}{10{,}521.33}$$

= 1.783 + 2.601 – 1.394 = 2.99 ksi

Compressive stress limit for pretensioned members at transfer stage is $0.6\,f'_{ci}$ [STD Art.9.15.2.1]

$$f'_{ci\text{-}reqd.} = \frac{2{,}990}{0.6} = 4{,}983.33 \text{ psi}$$

A.1.7.2.9 Initial Stresses at Girder End

The initial tensile stress at the top fiber and compressive stress at the bottom fiber of the girder at the girder end section are minimized by harping the web strands at the girder end. Following the TxDOT methodology (TxDOT 2001), the web strands are incrementally raised as a unit by two inches in each trial. The iterations are repeated until the top and bottom fiber stresses satisfies the allowable stress limits or the centroid of the topmost row of harped strands is at a distance of two inches from the top fiber of the girder in which case the concrete strength at release is updated based on the governing stress.

The position of the harped web strands, eccentricity of strands at the girder end, top and bottom fiber stresses at the girder end, and the corresponding required concrete strengths are summarized in Table A.1.7.1. The required concrete strengths are based on allowable stress limits at transfer stage specified in STD Art.9.15.2.1 presented as follows.

Allowable compressive stress limit = 0.6 f'_{ci}

For members with bonded reinforcement allowable tension at transfer = 7.5 $\sqrt{f'_{ci}}$

Table A.1.7.1. Summary of Top and Bottom Stresses at Girder End for Different Harped Strand Positions and Corresponding Required Concrete Strengths

Distance of the Centroid of Topmost Row of Harped Web Strands from		Eccentricity of Prestressing Strands at Girder End (in.)	Top Fiber Stress (psi)	Required Concrete Strength (psi)	Bottom Fiber Stress (psi)	Required Concrete Strength (psi)
Bottom Fiber (in.)	Top Fiber (in.)					
10 (no harping)	44	19.47	-1,291.11	29,634.91	4,383.73	7,306.22
12	42	19.07	-1,227.96	26,806.80	4,330.30	7,217.16
14	40	18.67	-1,164.81	24,120.48	4,276.86	7,128.10
16	38	18.27	-1,101.66	21,575.96	4,223.43	7,039.04
18	36	17.87	-1,038.51	19,173.23	4,169.99	6,949.99
20	34	17.47	-975.35	16,912.30	4,116.56	6,860.93
22	32	17.07	-912.20	14,793.17	4,063.12	6,771.87
24	30	16.67	-849.05	12,815.84	4,009.68	6,682.81
26	28	16.27	-785.90	10,980.30	3,956.25	6,593.75
28	26	15.87	-722.75	9,286.56	3,902.81	6,504.69
30	24	15.47	-659.60	7,734.62	3,849.38	6,415.63
32	22	15.07	-596.45	6,324.47	3,795.94	6,326.57
34	20	14.67	-533.30	5,056.12	3,742.51	6,237.51
36	18	14.27	-470.15	3,929.57	3,689.07	6,148.45
38	16	13.87	-407.00	2,944.82	3,635.64	6,059.39
40	14	13.47	-343.85	2,101.86	3,582.20	5,970.34
42	12	13.07	-280.69	1,400.70	3,528.77	5,881.28
44	10	12.67	-217.54	841.34	3,475.33	5,792.22
46	8	12.27	-154.39	423.77	3,421.89	5,703.16
48	6	11.87	-91.24	148.00	3,368.46	5,614.10
50	4	11.47	-28.09	14.03	3,315.02	5,525.04
52	2	11.07	35.06	58.43	3,261.59	5,435.98

From Table A.1.7.1, it is evident that the web strands are needed to be harped to the topmost position possible to control the bottom fiber stress at the girder end.

Detailed calculations for the case when 10 web strands (5 rows) are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of two inches from the top fiber of the girder) is presented as follows.

Eccentricity of prestressing strands at the girder end (see Figure A.1.7.2)

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 2(10) + 2(52+50+48+46+44)}{50}$$

$$= 11.07 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\, e_e}{S_t}$$

$$= \frac{1{,}405.53}{788.4} - \frac{1{,}405.53\,(11.07)}{8{,}902.67} = 1.783 - 1.748 = 0.035 \text{ ksi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\, e_e}{S_b}$$

$$f_{bi} = \frac{1{,}405.53}{788.4} + \frac{1{,}405.53\,(11.07)}{10{,}521.33} = 1.783 + 1.479 = 3.262 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is $0.6 f'_{ci}$ [STD Art.9.15.2.1]

$$f'_{ci\text{ -reqd.}} = \frac{3{,}262}{0.60} = 5{,}436.67 \text{ psi} \qquad \text{(controls)}$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release, f'_{ci} = 5,436.67 psi

Concrete strength at service, f'_c = 5,590 psi

A.1.7.3 Iteration 3

A third iteration is carried out to refine the prestress losses based on the updated concrete strengths. Based on the new prestress losses, the concrete strength at release and service will be further refined.

A.1.7.3.1 Concrete Shrinkage

[STD Art. 9.16.2.1.1]

For pretensioned members, the loss in prestress due to concrete shrinkage is given as

$$SH = 17{,}000 - 150\,RH \qquad \text{[STD Eq. 9-4]}$$

where:

RH is the relative humidity = 60%

$$SH = [17{,}000 - 150(60)]\frac{1}{1{,}000} = 8.0 \text{ ksi}$$

A.1.7.3.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

For pretensioned members, the loss in prestress due to elastic shortening is given as

$$ES = \frac{E_s}{E_{ci}} f_{cir} \qquad \text{[STD Eq. 9-6]}$$

where:

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

P_{si} = Pretension force after allowing for the initial losses, kips

As the initial losses are dependent on the elastic shortening and steel relaxation loss which are yet to be determined, the initial loss value of 9.27% obtained in the last trial (iteration 2) is taken as first estimate for the initial loss in prestress.

P_{si} = (number of strands)(area of strand)[0.9073(0.75 f_s')]
= 50(0.153)(0.9073)(0.75)(270) = 1,405.52 kips

M_g = Moment due to girder self-weight at midspan, k-ft.
= 1,209.98 k-ft.

e_c = Eccentricity of the prestressing strands at the midspan
= 19.47 in.

$$f_{cir} = \frac{1{,}405.52}{788.4} + \frac{1{,}405.52(19.47)^2}{260{,}403} - \frac{1{,}209.98(12 \text{ in./ft.})(19.47)}{260{,}403}$$

$$= 1.783 + 2.046 - 1.086 = 2.743 \text{ ksi}$$

Concrete strength at release, f'_{ci} = 5,436.67 psi

Modulus of elasticity of girder concrete at release is given as

$$E_{ci} = 33(w_c)^{3/2}\sqrt{f'_{ci}} \qquad \text{[STD Eq. 9-8]}$$

$$= [33(150)^{3/2}\sqrt{5{,}436.67}\,]\left(\frac{1}{1{,}000}\right) = 4{,}470.10 \text{ ksi}$$

Modulus of elasticity of prestressing steel, E_s = 28,000 ksi

Prestress loss due to elastic shortening is

$$ES = \left[\frac{28{,}000}{4{,}470.10}\right](2.743) = 17.18 \text{ ksi}$$

A.1.7.3.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

The loss in prestress due to creep of concrete is specified to be calculated using the following formula

$$CR_C = 12f_{cir} - 7f_{cds} \qquad \text{[STD Eq. 9-9]}$$

where:

$$f_{cds} = \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_{SDL} = Moment due to superimposed dead load at midspan section = 349.29 k-ft.

M_S = Moment due to slab weight at midspan section = 1,179.03 k-ft.

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

y_{bs} = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder = 24.75 – 19.47 = 5.28 in.

I = Moment of inertia of the non-composite section = 260,403 in.^{4}

I_c = Moment of inertia of composite section = 694,599.5 in.^{4}

$$f_{cds} = \frac{1{,}179.03(12 \text{ in./ft.})(19.47)}{260{,}403} + \frac{(349.29)(12 \text{ in./ft.})(41.157 - 5.28)}{694{,}599.5}$$

$$= 1.058 + 0.216 = 1.274 \text{ ksi}$$

Prestress loss due to creep of concrete is

$CR_C = 12(2.743) - 7(1.274) = 24.0$ ksi

A.1.7.3.4 Relaxation of Pretensioning Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strands, the prestress loss due to relaxation of the prestressing steel is calculated using the following formula

$$CR_S = 5{,}000 - 0.10\,ES - 0.05(SH + CR_C) \qquad \text{[STD Eq. 9-10A]}$$

$$CR_S = [5{,}000 - 0.10(17{,}180) - 0.05(8{,}000+24{,}000)]\left(\frac{1}{1{,}000}\right)$$

$= 1.682$ ksi

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75f'_s}$$

$$= \frac{[17.18 + 0.5(1.682)]100}{0.75(270)} = 8.90\% < 9.27\% \text{ (assumed value}$$

of initial prestress loss)

Therefore, another trial is required assuming 8.90% initial prestress loss.

The change in initial prestress loss will not affect the prestress loss due to concrete shrinkage. Therefore, the next trial will involve updating the losses due to elastic shortening, steel relaxation, and creep of concrete.

Based on an initial prestress loss value of 8.90%, the pretension force after allowing for the initial losses is calculated as follows.

P_{si} = (number of strands)(area of each strand)[0.911(0.75 f'_s)]

= 50(0.153)(0.911)(0.75)(270) = 1,411.25 kips

Loss in prestress due to elastic shortening

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$= \frac{1{,}411.25}{788.4} + \frac{1{,}411.25(19.47)^2}{260{,}403} - \frac{1{,}209.98(12 \text{ in./ft.})(19.47)}{260{,}403}$$

$= 1.790 + 2.054 - 1.086 = 2.758$ ksi

E_s = 28,000 ksi

E_{ci} = 4,470.10 ksi

$$ES = \left[\frac{28{,}000}{4{,}470.10}\right](2.758) = 17.28 \text{ ksi}$$

Loss in prestress due to creep of concrete

$CR_C = 12f_{cir} - 7f_{cds}$

The value of f_{cds} is independent of the initial prestressing force value and will be same as calculated in Section A.1.7.3.3.

f_{cds} = 1.274 ksi

CR_C = 12(2.758) – 7(1.274) = 24.18 ksi

Loss in prestress due to relaxation of steel

$CR_S = 5{,}000 - 0.10\,ES - 0.05(SH + CR_C)$

$$= [5{,}000 - 0.10(17{,}280) - 0.05(8{,}000 + 24{,}180)]\left(\frac{1}{1{,}000}\right)$$

= 1.663 ksi

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2}CR_S)100}{0.75f_s'}$$

$$= \frac{[17.28 + 0.5(1.663)]100}{0.75(270)} = 8.94\% \approx 8.90\% \text{ (assumed value}$$

for initial prestress loss)

A.1.7.3.5 Total Losses at Transfer

Total prestress loss at transfer = $(ES + \frac{1}{2}CR_s)$

= [17.28 + 0.5(1.663)] = 18.11 ksi

Effective initial prestress, f_{si} = 202.5 – 18.11 = 184.39 ksi

P_{si} = Effective pretension after allowing for the initial prestress loss

= (number of strands)(area of strand)(f_{si})

= 50(0.153)(184.39) = 1,410.58 kips

A.1.7.3.6 Total Losses at Service Loads

Loss in prestress due to concrete shrinkage, SH = 8.0 ksi

Loss in prestress due to elastic shortening, ES = 17.28 ksi

Loss in prestress due to creep of concrete, CR_C = 24.18 ksi

Loss in prestress due to steel relaxation, CR_S = 1.663 ksi

Total final loss in prestress = $SH + ES + CR_C + CR_S$

= 8.0 + 17.28 + 24.18 + 1.663 = 51.12 ksi

or $\dfrac{51.12(100)}{0.75(270)} = 25.24\ \%$

Effective final prestress, f_{se} = 0.75(270) – 51.12 = 151.38 ksi

P_{se} = Effective pretension after allowing for the final prestress loss

= (number of strands)(area of strand)(effective final prestress)

= 50(0.153)(151.38) = 1,158.06 kips

A.1.7.3.7 Final Stresses at Midspan

Concrete stress at top fiber of the girder at midspan section due to applied loads and effective prestress

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1{,}158.06}{788.4} - \frac{1{,}158.06\,(19.47)}{8{,}902.67} + 3.626$$

$$= 1.469 - 2.533 + 3.626 = 2.562 \text{ ksi}$$

(f_t calculations are presented in Section A.1.6.1)

Compressive stress limit under service load combination is $0.6\,f_c'$

[STD Art. 9.15.2.2]

$$f_c'\text{-reqd.} = \frac{2{,}562}{0.6} = 4{,}270 \text{ psi}$$

Concrete stress at top fiber of the girder at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

$$= \frac{1{,}158.06}{788.4} - \frac{1{,}158.06\,(19.47)}{8{,}902.67} + \frac{(1{,}209.98 + 1{,}179.03)(12 \text{ in./ft.})}{8{,}902.67} + \frac{349.29(12 \text{ in./ft.})}{54{,}083.9}$$

$$= 1.469 - 2.533 + 3.22 + 0.077 = 2.233 \text{ ksi}$$

Compressive stress limit for effective prestress + permanent dead loads = $0.4\,f_c'$ [STD Art. 9.15.2.2]

$$f_c'\text{-reqd.} = \frac{2{,}233}{0.40} = 5{,}582.5 \text{ psi} \quad \text{(controls)}$$

Concrete stress at top fiber of the girder at midspan due to live load + ½(effective prestress + dead loads)

$$f_{tf} = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}\right)$$

$$= \frac{1,478.39(12\text{ in./ft.})}{54,083.9} + 0.5\left\{\frac{1,158.06}{788.4} - \frac{1,158.06\,(19.47)}{8,902.67} + \frac{(1,209.98\ +\ 1,179.03)(12\text{ in./ft.})}{8,902.67} + \frac{349.29(12\text{ in./ft.})}{54,083.9}\right\}$$

$$= 0.328 + 0.5(1.469 - 2.533 + 3.22 + 0.077) = 1.445\text{ ksi}$$

Allowable limit for compressive stress due to live load + ½(effective prestress + dead loads) = 0.4 f'_c [STD Art. 9.15.2.2]

$$f'_c\text{-}reqd. = \frac{1,445}{0.40} = 3,612.5\text{ psi}$$

Tensile stress at the bottom fiber of the girder at midspan due to service loads

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - f_b$$ (f_b calculations are presented in Sec. A.1.6.1)

$$= \frac{1,158.06}{788.4} + \frac{1,158.06\,(19.47)}{10,521.33} - 4.024$$

$= 1.469 + 2.143 - 4.024 = -\,0.412$ ksi (negative sign indicates tensile stress)

For members with bonded reinforcement allowable tension in the precompressed tensile zone = $6\sqrt{f'_c}$ [STD Art. 9.15.2.2]

$$f'_c\text{-}reqd. = \left(\frac{412}{6}\right)^2 = 4,715.1\text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations. The required concrete strength at service is determined to be 5,582.5 psi.

A.1.7.3.8
Initial Stresses at Hold Down Point

Prestressing force after allowing for initial prestress loss

P_{si} = (number of strands)(area of strand)(effective initial prestress)

= 50(0.153)(184.39) = 1410.58 kips (Effective initial prestress calculations are presented in Section A.1.7.3.5)

Initial concrete stress at top fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\,e_c}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at hold down point based on overall girder length of 109'-8"
= 1,222.22 k-ft. (see Section A.1.7.1.8)

$$f_{ti} = \frac{1{,}410.58}{788.4} - \frac{1{,}410.58\,(19.47)}{8{,}902.67} + \frac{1{,}222.22(12\text{ in./ft.})}{8{,}902.67}$$

$$= 1.789 - 3.085 + 1.647 = 0.351 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1{,}410.58}{788.4} + \frac{1{,}410.58\,(19.47)}{10{,}521.33} - \frac{1{,}222.22(12\text{ in./ft.})}{10{,}521.33}$$

$$= 1.789 + 2.610 - 1.394 = 3.005 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is $0.6\,f'_{ci}$ [STD Art.9.15.2.1]

$$f'_{ci}\text{-reqd.} = \frac{3{,}005}{0.6} = 5{,}008.3 \text{ psi}$$

A.1.7.3.9
Initial Stresses at Girder End

The eccentricity of the prestressing strands at the girder end when 10 web strands are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of two inches from the top fiber of the girder) is calculated as follows (see Fig. A.1.7.2)

$$e_e = 24.75 - \frac{10(2{+}4{+}6) + 8(8) + 2(10) + 2(52{+}50{+}48{+}46{+}44)}{50}$$

$$= 11.07 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\, e_e}{S_t}$$

$$= \frac{1{,}410.58}{788.4} - \frac{1{,}410.58\,(11.07)}{8{,}902.67} = 1.789 - 1.754 = 0.035 \text{ ksi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\, e_e}{S_b}$$

$$f_{bi} = \frac{1{,}410.58}{788.4} + \frac{1{,}410.58\,(11.07)}{10{,}521.33} = 1.789 + 1.484 = 3.273 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is $0.6\, f'_{ci}$ [STD Art.9.15.2.1]

$$f'_{ci}\text{-}reqd. = \frac{3{,}273}{0.60} = 5{,}455 \text{ psi (controls)}$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release, f'_{ci} = 5,455 psi

Concrete strength at service, f'_c = 5,582.5 psi

The difference in the required concrete strengths at release and at service obtained from iterations 2 and 3 is less than 20 psi. Hence the concrete strengths are sufficiently converged and another iteration is not required.

Therefore provide f'_{ci} = 5,455 psi

f'_c = 5,582.5 psi

50 – ½ in. diameter, 10 draped at the end, GR 270 low-relaxation strands.

The final strand patterns at the midspan section and at the girder ends are shown in Figures A.1.7.1 and A.1.7.2. The longitudinal strand profile is shown in Figure A.1.7.3.

No. of Strands	Distance from Bottom Fiber (in.)
4	10
10	8
12	6
12	4
12	2

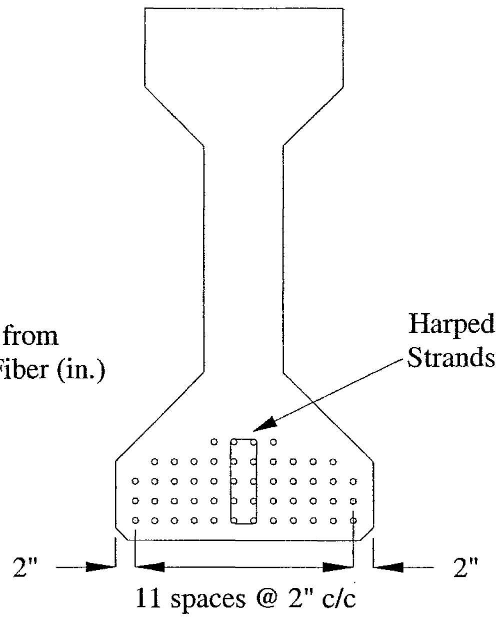

Figure A.1.7.1. Final Strand Pattern at Midspan

No. of Strands	Distance from Bottom Fiber (in.)
2	52
2	50
2	48
2	46
2	44

No. of Strands	Distance from Bottom Fiber (in.)
2	10
8	8
10	6
10	4
10	2

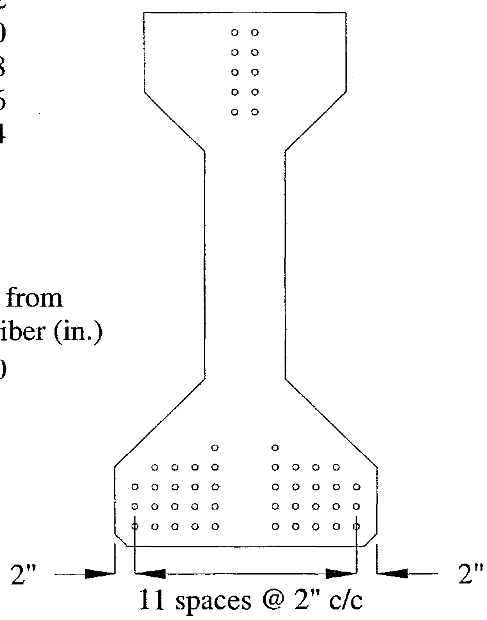

Figure A.1.7.2. Final Strand Pattern at Girder End

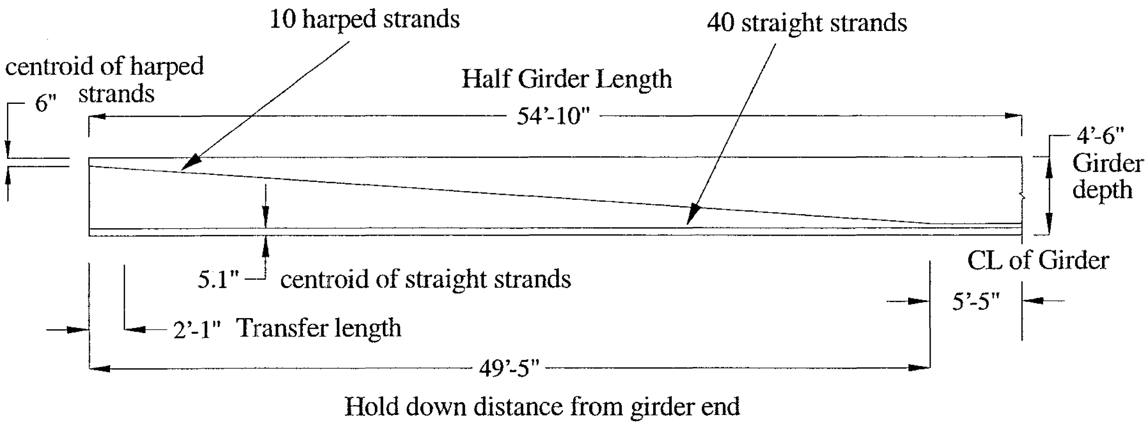

Figure A.1.7.3 Longitudinal Strand Profile (half of the girder length is shown)

The distance between the centroid of the 10 harped strands and the top fiber of the girder at the girder end

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

The distance between the centroid of the 10 harped strands and the bottom fiber of the girder at the harp points

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

Transfer length distance from girder end = 50 (strand diameter) [STD Art. 9.20.2.4]

Transfer length = 50(0.50) = 25 in. = 2.083 ft.

The distance between the centroid of the 10 harped strands and the top of the girder at the transfer length section

$$= 6 \text{ in.} + \frac{(54 \text{ in} - 6 \text{ in} - 6 \text{ in})}{49.4 \text{ ft.}}(2.083 \text{ ft.}) = 7.77 \text{ in.}$$

The distance between the centroid of the 40 straight strands and the bottom fiber of the girder at all locations

$$= \frac{10(2) + 10(4) + 10(6) + 8(8) + 2(10)}{40} = 5.1 \text{ in.}$$

A.1.8 STRESS SUMMARY

A.1.8.1 Concrete Stresses at Transfer

A.1.8.1.1 Allowable Stress Limits

[STD Art. 9.15.2.1]

The allowable stress limits at transfer specified by the Standard Specifications are as follows.

Compression: $0.6\,f'_{ci}$ = 0.6(5,455) = +3,273 psi = 3.273 ksi (comp.)

Tension: The maximum allowable tensile stress is

$7.5\sqrt{f'_{ci}} = 7.5\sqrt{5{,}455} = -553.93$ psi (tension)

If the calculated tensile stress exceeds 200 psi or $3\sqrt{f'_{ci}} = 3\sqrt{5{,}455} = 221.57$ psi, whichever is smaller, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

A.1.8.1.2 Stresses at Girder End

Stresses at the girder end are checked only at transfer, because it almost always governs.

Eccentricity of prestressing strands at the girder end when 10 web strands are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of two inches from the top fiber of the girder)

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 2(10) + 2(52+50+48+46+44)}{50}$$

$$= 11.07 \text{ in.}$$

Prestressing force after allowing for initial prestress loss

P_{si} = (number of strands)(area of strand)(effective initial prestress)

= 50(0.153)(184.39) = 1,410.58 kips (Effective initial prestress calculations are presented in Section A.1.7.3.5)

Concrete stress at the top fiber of the girder at the girder end at transfer:

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\, e_e}{S_t}$$

$$= \frac{1{,}410.58}{788.4} - \frac{1{,}410.58\,(11.07)}{8{,}902.67} = 1.789 - 1.754 = +0.035 \text{ ksi}$$

Allowable Compression: +3.273 ksi >> +0.035 ksi (reqd.) (O.K.)

Because the top fiber stress is compressive, there is no need for additional bonded reinforcement.

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\, e_e}{S_b}$$

$$= \frac{1{,}410.58}{788.4} + \frac{1{,}410.58\ (11.07)}{10{,}521.33} = 1.789 + 1.484 = +3.273 \text{ ksi}$$

Allowable compression: +3.273 ksi = +3.273 ksi (reqd.) (O.K.)

A.1.8.1.3 Stresses at Transfer Length Section

Stresses at transfer length are checked only at release, because it almost always governs.

Transfer length = 50(strand diameter) [STD Art. 9.20.2.4]
= 50 (0.50) = 25 in. = 2.083 ft.

The transfer length section is located at a distance of 2'-1" from the end of the girder or at a point 1'-6.5" from the centerline of the bearing as the girder extends 6.5 in. beyond the bearing centerline. Overall girder length of 109'-8" is considered for the calculation of bending moment at transfer length.

Moment due to girder self-weight, $M_g = 0.5wx(L - x)$

where:

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Transfer length distance from girder end = 2.083 ft.

$M_g = 0.5(0.821)(2.083)(109.67 - 2.083) = 92$ k–ft.

Eccentricity of prestressing strands at transfer length section

$$e_t = e_c - (e_c - e_e)\ \frac{(49.404 - x)}{49.404}$$

where:

e_c = Eccentricity of prestressing strands at midspan = 19.47 in.

e_e = Eccentricity of prestressing strands at girder end = 11.07 in.

x = Distance of transfer length section from girder end, ft.

$$e_t = 19.47 - (19.47 - 11.07)\frac{(49.404 - 2.083)}{49.404} = 11.42 \text{ in.}$$

Initial concrete stress at top fiber of the girder at transfer length section due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\, e_t}{S_t} + \frac{M_g}{S_t}$$

$$f_{ti} = \frac{1{,}410.58}{788.4} - \frac{1{,}410.58(11.42)}{8{,}902.67} + \frac{92\,(12\,\text{in./ft.})}{8{,}902.67}$$

$$= 1.789 - 1.809 + 0.124 = +0.104 \text{ ksi}$$

Allowable compression: +3.273 ksi >> 0.104 ksi (reqd.) (O.K.)

Because the top fiber stress is compressive, there is no need for additional bonded reinforcement.

Initial concrete stress at bottom fiber of the girder at transfer length section due to self-weight of girder and effective initial prestress.

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\, e_t}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1{,}410.58}{788.4} + \frac{1{,}410.58(11.42)}{10{,}521.33} - \frac{92\,(12\,\text{in./ft.})}{10{,}521.33}$$

$$= 1.789 + 1.531 - 0.105 = 3.215 \text{ ksi}$$

Allowable compression: +3.273 ksi > 3.215 ksi (reqd.) (O.K.)

A.1.8.1.4 Stresses at Hold Down Points

The eccentricity of the prestressing strands at the harp points is the same as at midspan.
$e_{harp} = e_c = 19.47$ in.

Initial concrete stress at top fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\, e_{harp}}{S_t} + \frac{M_g}{S_t}$$

where:

M_g = Moment due to girder self-weight at hold down point based on overall girder length of 109'-8"
= 1,222.22 k-ft. (see Section A.1.7.1.8)

$$f_{ti} = \frac{1,410.58}{788.4} - \frac{1,410.58(19.47)}{8,902.67} + \frac{1,222.22(12 \text{ in./ft.})}{8,902.67}$$

$$= 1.789 - 3.085 + 1.647 = 0.351 \text{ ksi}$$

Allowable compression: +3.273 ksi >> 0.351 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at hold down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\, e_{harp}}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1,410.58}{788.4} + \frac{1,410.58(19.47)}{10,521.33} - \frac{1,222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.789 + 2.610 - 1.394 = 3.005 \text{ ksi}$$

Allowable compression: +3.273 ksi > 3.005 ksi (reqd.) (O.K.)

A.1.8.1.5 Stresses at Midspan

Bending moment due to girder self-weight at midspan section based on overall girder length of 109'-8"

$$M_g = 0.5wx(L - x)$$

where:

w = Self-weight of the girder = 0.821 kips/ft.

L = Overall girder length = 109.67 ft.

x = Half the girder length = 54.84 ft.

$$M_g = 0.5(0.821)(54.84)(109.67 - 54.84) = 1,234.32 \text{ k–ft.}$$

Initial concrete stress at top fiber of the girder at midspan section due to self-weight of the girder and the effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\, e_c}{S_t} + \frac{M_g}{S_t}$$

$$f_{ti} = \frac{1,410.58}{788.4} - \frac{1,410.58(19.47)}{8,902.67} + \frac{1,234.32(12 \text{ in./ft.})}{8,902.67}$$

$$= 1.789 - 3.085 + 1.664 = 0.368 \text{ ksi}$$

Allowable compression: +3.273 ksi >> 0.368 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at midspan section due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1{,}410.58}{788.4} + \frac{1{,}410.58(19.47)}{10{,}521.33} - \frac{1{,}234.32(12\text{ in./ft.})}{10{,}521.33}$$

$$= 1.789 + 2.610 - 1.408 = 2.991 \text{ ksi}$$

Allowable compression: +3.273 ksi > 2.991 ksi (reqd.) (O.K.)

A.1.8.1.6 Stress Summary at Transfer

Allowable Stress Limits:

Compression: + 3.273 ksi

Tension: – 0.20 ksi without additional bonded reinforcement
– 0.554 ksi with additional bonded reinforcement

Location	Top of girder f_t (ksi)	Bottom of girder f_b (ksi)
Girder end	+0.035	+3.273
Transfer length section	+0.104	+3.215
Hold down points	+0.351	+3.005
Midspan	+0.368	+2.991

A.1.8.2 Concrete Stresses at Service Loads

A.1.8.2.1 Allowable Stress Limits

[STD Art. 9.15.2.2]

The allowable stress limits at service load after losses have occurred specified by the Standard Specifications are presented as follows.

Compression:

Case (I): For all load combinations

$0.60\,f_c' = 0.60(5{,}582.5)/1{,}000 = +3.349$ ksi (for precast girder)

$0.60\,f_c' = 0.60(4{,}000)/1{,}000 = +2.400$ ksi (for slab)

Case (II): For effective prestress + permanent dead loads

$0.40\,f_c' = 0.40(5{,}582.5)/1000 = +2.233$ ksi (for precast girder)

$0.40\,f_c' = 0.40(4{,}000)/1{,}000 = +1.600$ ksi (for slab)

Case (III): For live loads +1/2(effective prestress + dead loads)

$0.40 f'_c = 0.40(5{,}582.5)/1{,}000 = +2.233$ ksi (for precast girder)

$0.40 f'_c = 0.40(4{,}000)/1{,}000 = +1.600$ ksi (for slab)

Tension: For members with bonded reinforcement

$$6\sqrt{f'_c} = 6\sqrt{5{,}582.5}\left(\frac{1}{1{,}000}\right) = -0.448 \text{ ksi}$$

A.1.8.2.2 Final Stresses at Midspan

Effective pretension after allowing for the final prestress loss

P_{se} = (number of strands)(area of strand)(effective final prestress)

= 50(0.153)(151.38) = 1,158.06 kips

Case (I): Service load conditions

Concrete stress at the top fiber of the girder at the midspan section due to service loads and effective prestress

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}}$$

$$= \frac{1{,}158.06}{788.4} - \frac{1{,}158.06\,(19.47)}{8{,}902.67} + \frac{(1{,}209.98 + 1{,}179.03)(12 \text{ in./ft.})}{8{,}902.67}$$

$$+ \frac{(349.29 + 1{,}478.39)(12 \text{ in./ft.})}{54{,}083.9}$$

= 1.469 – 2.533 + 3.220 + 0.406 = 2.562 ksi

Allowable compression: +3.349 ksi > +2.562 ksi (reqd.) (O.K.)

Case (II): Effective prestress + permanent dead loads

Concrete stress at top fiber of the girder at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

$$= \frac{1{,}158.06}{788.4} - \frac{1{,}158.06\,(19.47)}{8{,}902.67} + \frac{(1{,}209.98 + 1{,}179.03)(12 \text{ in./ft.})}{8{,}902.67}$$

$$+ \frac{349.29(12 \text{ in./ft.})}{54{,}083.9}$$

= 1.469 – 2.533 + 3.22 + 0.077 = 2.233 ksi

Allowable compression: +2.233 ksi = +2.233 ksi (reqd.) (O.K.)

Case (III): Live loads + ½(prestress + dead loads)

Concrete stress at top fiber of the girder at midspan due to live load + ½(effective prestress + dead loads)

$$f_{tf} = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}\right)$$

$$= \frac{1{,}478.39(12\text{ in./ft.})}{54{,}083.9} + 0.5\left\{\frac{1{,}158.06}{788.4} - \frac{1{,}158.06\,(19.47)}{8{,}902.67} + \frac{(1{,}209.98 + 1{,}179.03)(12\text{ in./ft.})}{8{,}902.67} + \frac{349.29(12\text{ in./ft.})}{54{,}083.9}\right\}$$

$$= 0.328 + 0.5(1.469 - 2.533 + 3.22 + 0.077) = 1.445 \text{ ksi}$$

Allowable compression: +2.233 ksi > +1.445 ksi (reqd.) (O.K.)

Tensile stress at the bottom fiber of the girder at midspan due to service loads

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - \frac{M_g + M_S}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

$$= \frac{1{,}158.06}{788.4} + \frac{1{,}158.06\,(19.47)}{10{,}521.33} - \frac{(1{,}209.98 + 1{,}179.03)(12\text{ in./ft.})}{10{,}521.33} - \frac{(349.29 + 1{,}478.39)(12\text{ in./ft.})}{16{,}876.83}$$

$= 1.469 + 2.143 - 2.725 - 1.299 = -0.412$ ksi (negative sign indicates tensile stress)

Allowable Tension: –0.448 ksi < –412 ksi (reqd.) (O.K.)

Superimposed dead and live loads contribute to the stresses at the top of the slab calculated as follows

Case (I): Superimposed dead load and live load effect

Concrete stress at top fiber of the slab at midspan due to live load + superimposed dead loads

$$f_t = \frac{M_{SDL} + M_{LL+I}}{S_{tc}} = \frac{(349.29 + 1{,}478.39)(12\text{ in./ft.})}{33{,}325.31} = +0.658 \text{ ksi}$$

Allowable compression: +2.400 ksi > +0.658 ksi (reqd.) (O.K.)

Case (II): Superimposed dead load effect

Concrete stress at top fiber of the slab at midspan due to superimposed dead loads

$$f_t = \frac{M_{SDL}}{S_{tc}} = \frac{(349.29)(12\text{ in./ft.})}{33,325.31} = 0.126\text{ ksi}$$

Allowable compression: +1.600 ksi > +0.126 ksi (reqd.) (O.K.)

Case (III): Live load + 0.5(superimposed dead loads)

Concrete stress at top fiber of the slab at midspan due to live loads + 0.5(superimposed dead loads)

$$f_t = \frac{M_{LL+I} + 0.5(M_{SDL})}{S_{tc}}$$

$$= \frac{(1{,}478.39)(12\text{ in./ft.}) + 0.5(349.29)(12\text{ in./ft.})}{33{,}325.31} = 0.595\text{ ksi}$$

Allowable compression: +1.600 ksi > +0.595 ksi (reqd.) (O.K.)

A.1.8.2.3 Summary of Stresses at Service Loads

At Midspan	Top of slab f_t (ksi)	Top of Girder f_t (ksi)	Bottom of girder f_b (ksi)
Case I	+0.658	+2.562	– 0.412
Case II	+0.126	+2.233	–
Case III	+0.595	+1.455	–

A.1.8.2.4 Composite Section Properties

The composite section properties calculated in Section A.1.4.2.4 were based on the modular ratio value of 1. But as the actual concrete strength is now selected, the actual modular ratio can be determined and the corresponding composite section properties can be evaluated.

Modular ratio between slab and girder concrete

$$n = \left(\frac{E_{cs}}{E_{cp}}\right)$$

where:

n = Modular ratio between slab and girder concrete

E_{cs} = Modulus of elasticity of slab concrete, ksi

$= 33(w_c)^{3/2}\sqrt{f'_{cs}}$ [STD Eq. 9-8]

w_c = Unit weight of concrete = 150 pcf

f'_{cs} = Compressive strength of slab concrete at service
= 4,000 psi

$$E_{cs} = [33(150)^{3/2}\sqrt{4000}\,]\left(\frac{1}{1000}\right) = 3{,}834.25 \text{ ksi}$$

E_{cp} = Modulus of elasticity of precast girder concrete, ksi
$= 33(w_c)^{3/2}\sqrt{f'_c}$

f'_c = Compressive strength of precast girder concrete at service
= 5,582.5 psi

$$E_{cp} = [33(150)^{3/2}\sqrt{5{,}582.5}\,]\left(\frac{1}{1{,}000}\right) = 4{,}529.65 \text{ ksi}$$

$$n = \frac{3{,}834.25}{4{,}529.65} = 0.846$$

Transformed flange width, $b_{tf} = n$*(effective flange width)

Effective flange width = 96 in. (see Section A.1.4.2)

b_{tf} = 0.846*(96) = 81.22 in.

Transformed Flange Area, $A_{tf} = n$*(effective flange width)(t_s)

t_s = Slab thickness = 8 in.

A_{tf} = 0.846*(96)(8) = 649.73 in.^{2}

Table A.1.8.1. Properties of Composite Section

	Transformed Area A (in.^{2})	y_b in.	Ay_b in.^{3}	$A(y_{bc} - y_b)^2$	I in.^{4}	$I + A(y_{bc} - y_b)^2$ in.^{4}
Girder	788.40	24.75	19,512.9	177,909.63	260,403.0	438,312.6
Slab	649.73	58.00	37,684.3	215,880.37	3,465.4	219,345.8
Σ	1,438.13		57,197.2			657,658.4

A_c = Total area of composite section = 1,438.13 in.^{2}

h_c = Total height of composite section = 54 in. + 8 in. = 62 in.

I_c = Moment of inertia of composite section = 657,658.4 in^4

y_{bc} = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.
= 57,197.2/1438.13 = 39.77 in.

y_{tg} = Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.
= 54 - 39.772 = 14.23 in.

y_{tc} = Distance from the centroid of the composite section to extreme top fiber of the slab = 62 - 39.77 = 22.23 in.

S_{bc} = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, $in.^3$
= I_c/y_{bc} = 657,658.4/39.77 = 16,535.71 $in.^3$

S_{tg} = Section modulus of composite section referenced to the top fiber of the precast girder, $in.^3$
= I_c/y_{tg} = 657,658.4/14.23 = 46,222.83 $in.^3$

S_{tc} = Section modulus of composite section referenced to the top fiber of the slab, $in.^3$
= I_c/y_{tc} = 657,658.4/22.23 = 29,586.93 $in.^3$

A.1.9 FLEXURAL STRENGTH

[STD Art. 9.17]

The flexural strength limit state is investigated for Group I loading as follows

The Group I load factor design combination specified by the Standard Specifications is

$M_u = 1.3[M_g + M_S + M_{SDL} + 1.67(M_{LL+I})]$ [STD Table 3.22.1.A]

where:

M_u = Design flexural moment at midspan of the girder, k-ft.

M_g = Moment due to self-weight of the girder at midspan
= 1,209.98 k-ft.

M_S = Moment due to slab weight at midspan = 1,179.03 k-ft.

M_{SDL} = Moment due to superimposed dead loads at midspan
= 349.29 k-ft.

M_{LL+I} = Moment due to live loads including impact loads at midspan = 1,478.39 k-ft.

Substituting the moment values from Table A.1.5.1 and A.1.5.2

$M_u = 1.3[1{,}209.98 + 1{,}179.03 + 349.29 + 1.67(1{,}478.39)]$

$= 6{,}769.37$ k-ft.

For bonded members, the average stress in the pretensioning steel at ultimate load conditions is given as

$$f^*_{su} = f'_s\left(1 - \frac{\gamma^*}{\beta_1}\rho^*\frac{f'_s}{f'_c}\right) \quad \text{[STD Eq. 9-17]}$$

The above equation is applicable when the effective prestress after losses, $f_{se} > 0.5\, f'_s$

where:

f^*_{su} = Average stress in the pretensioning steel at ultimate load, ksi

f'_s = Ultimate Stress in prestressing strands = 270 ksi

f_{se} = Effective final prestress (see Section A.1.7.3.6)
= 151.38 ksi > 0.5 (270) = 135 ksi (O.K.)
The equation for f^*_{su} shown above is applicable.

f'_c = Compressive strength of slab concrete at service
= 4,000 psi

γ^* = Factor for type of prestressing steel
= 0.28 for low-relaxation steel strands [STD Art. 9.1.2]

$\beta_1 = 0.85 - 0.05\dfrac{(f'_c - 4{,}000)}{1{,}000} \geq 0.65$ [STD Art. 8.16.2.7]

It is assumed that the neutral axis lies in the slab, and hence the f'_c of slab concrete is used for the calculation of the factor β_1. If the neutral axis is found to be lying below the slab, β_1 will be updated.

$\beta_1 = 0.85 - 0.05\dfrac{(4{,}000 - 4{,}000)}{1{,}000} = 0.85$

ρ^* = Ratio of prestressing steel = $\dfrac{A^*_s}{b\,d}$

A_s^* = Area of pretensioned reinforcement, in.^{2}
= (number of strands)(area of strand) = 50(0.153) = 7.65 in.^{2}

b = Effective flange (composite slab) width = 96 in.

y_{bs} = Distance from centroid of the strands to the bottom fiber of the girder at midspan = 5.28 in. (see Section A.1.7.3.3)

d = Distance from top of the slab to the centroid of prestressing strands, in.
= girder depth (h) + slab thickness (t_s) – y_{bs}
= 54 + 8 – 5.28 = 56.72 in.

$$\rho^* = \frac{7.65}{96(56.72)} = 0.001405$$

$$f_{su}^* = 270\left[1-\left(\frac{0.28}{0.85}\right)(0.001405)\left(\frac{270.0}{4.0}\right)\right] = 261.565 \text{ ksi}$$

Depth of equivalent rectangular compression block

$$a = \frac{A_s^* f_{su}^*}{0.85 f_c' b} = \frac{7.65(261.565)}{0.85(4)(96)}$$

= 6.13 in. < t_s = 8.0 in. [STD Art. 9.17.2]

The depth of compression block is less than the flange (slab) thickness. Hence, the section is designed as a rectangular section and f_c' of the slab concrete is used for calculations.

For rectangular section behavior, the design flexural strength is given as

$$\phi M_n = \phi\left[A_s^* f_{su}^* d\left(1-0.6\frac{\rho^* f_{su}^*}{f_c'}\right)\right]$$ [STD Eq. 9-13]

where:

ϕ = Strength reduction factor = 1.0 for prestressed concrete members [STD Art. 9.14]

M_n = Nominal moment strength of the section

$$\phi M_n = 1.0\left[(7.65)(261.565)\frac{(56.72)}{(12 \text{ in./ft.})}\left(1-0.6\frac{0.001405(261.565)}{4.0}\right)\right]$$

= 8,936.56 k-ft. > M_u = 6,769.37 k-ft. (OK)

A.1.10 DUCTILITY LIMITS

[STD Art. 9.18]

A.1.10.1 Maximum Reinforcement

[STD Art. 9.18.1]

To ensure that steel is yielding as ultimate capacity is approached, the reinforcement index for a rectangular section shall be such that

$$\frac{\rho^* f_{su}^*}{f_c'} < 0.36\beta_1 \qquad \text{[STD Eq. 9.20]}$$

$$0.001405\left(\frac{261.565}{4.0}\right) = 0.092 < 0.36(0.85) = 0.306 \qquad \text{(O.K.)}$$

A.1.10.2 Minimum Reinforcement

[STD Art. 9.18.2]

The nominal moment strength developed by the prestressed and nonprestressed reinforcement at the critical section shall be at least 1.2 times the cracking moment, M_{cr}^*

$$\phi M_n \geq 1.2\ M_{cr}^*$$

$$M_{cr}^* = (f_r + f_{pe})\,S_{bc} - M_{d\text{-}nc}\left(\frac{S_{bc}}{S_b} - 1\right) \qquad \text{[STD Art. 9.18.2.1]}$$

where:

f_r = Modulus of rupture of concrete = $7.5\sqrt{f_c'}$ for normal weight concrete, ksi [STD Art. 9.15.2.3]

$$= 7.5\sqrt{5{,}582.5}\left(\frac{1}{1{,}000}\right) = 0.5604 \text{ ksi}$$

f_{pe} = Compressive stress in concrete due to effective prestress forces only at extreme fiber of section where tensile stress is caused by externally applied loads, ksi

The tensile stresses are caused at the bottom fiber of the girder under service loads. Therefore f_{pe} is calculated for the bottom fiber of the girder as follows.

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

P_{se} = Effective prestress force after losses = 1,158.06 kips

e_c = Eccentricity of prestressing strands at midspan = 19.47 in.

$$f_{pe} = \frac{1{,}158.06}{788.4} + \frac{1{,}158.06(19.47)}{10{,}521.33} = 1.469 + 2.143 = 3.612 \text{ ksi}$$

$M_{d\text{-}nc}$ = Non-composite dead load moment at midspan due to self-weight of girder and weight of slab
= 1,209.98 + 1,179.03 = 2,389.01 k-ft. = 28,668.12 k-in.

S_b = Section modulus of the precast section referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

S_{bc} = Section modulus of the composite section referenced to the extreme bottom fiber of the precast girder
= 16,535.71 in.^{3}

$$M^*_{cr} = (0.5604 + 3.612)(16{,}535.71) - (28{,}668.12)\left(\frac{16{,}535.71}{10{,}521.33} - 1\right)$$

$$= 68{,}993.6 - 16{,}387.8 = 52{,}605.8 \text{ k-in.} = 4{,}383.8 \text{ k-ft.}$$

$1.2M^*_{cr} = 1.2(4{,}383.8) = 5{,}260.56$ k-ft. $< \phi M_n = 8{,}936.56$ k-ft.
(O.K.)

A.1.11 SHEAR DESIGN

[STD Art. 9.20]

The shear design for the AASHTO Type IV girder based on the Standard Specifications is presented in the following section.

Prestressed concrete members subject to shear shall be designed so that

$$V_u < \phi(V_c + V_s) \quad \text{[STD Eq. 9-26]}$$

where:

V_u = Factored shear force at the section considered (calculated using load combination causing maximum shear force), kips

V_c = Nominal shear strength provided by concrete, kips

V_s = Nominal shear strength provided by web reinforcement, kips

ϕ = Strength reduction factor for shear = 0.90 for prestressed concrete members [STD Art. 9.14]

The critical section for shear is located at a distance $h/2$ (h is the depth of composite section) from the face of the support. However as the support dimensions are unknown, the critical section for shear is conservatively calculated from the centerline of the bearing support. [STD Art. 9.20.1.4]

Distance of critical section for shear from bearing centerline

$= h/2 = \dfrac{62}{2(12 \text{ in./ft.})} = 2.583$ ft.

From Tables A.1.5.1 and A.1.5.2 the shear forces at the critical section are as follows

V_d = Shear force due to total dead load at the critical section = 96.07 kips

V_{LL+I} = Shear force due to live load including impact at critical section = 56.60 kips

The shear design is based on Group I loading presented as follows.

Group I load factor design combination specified by the Standard Specifications is

$V_u = 1.3(V_d + 1.67\ V_{LL+I}) = 1.3(96.07 + 1.67(56.6)) = 247.8$ kips

Shear strength provided by normal weight concrete, V_c, shall be taken as the lesser of the values V_{ci} or V_{cw}. [STD Art. 9.20.2]

Computation of V_{ci} [STD Art. 9.20.2.2]

$$V_{ci} = 0.6\sqrt{f_c'}b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.7\sqrt{f_c'}b'd \qquad \text{[STD Eq. 9-27]}$$

where

V_{ci} = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, kips

f_c' = Compressive strength of girder concrete at service = 5,582.5 psi

b' = Width of the web of a flanged member = 8 in.

d = Distance from the extreme compressive fiber to centroid of pretensioned reinforcement, but not less than $0.8h_c$
$= h_c - (y_b - e_x)$ [STD Art. 9.20.2.2]

h_c = Depth of composite section = 62 in.

y_b = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

e_x = Eccentricity of prestressing strands at the critical section for shear

$$= e_c - (e_c - e_e)\ \frac{(49.404 - x)}{49.404}$$

e_c = Eccentricity of prestressing strands at midspan
= 19.12 in.

e_e = Eccentricity of prestressing strands at the girder end
= 11.07 in.

x = Distance of critical section from girder end = 2.583 ft.

$$e_x = 19.47 - (19.47 - 11.07)\frac{(49.404 - 2.583)}{49.404} = 11.51 \text{ in.}$$

d = 62 – (24.75 – 11.51) = 48.76 in.
= $0.8h_c$ = 0.8(62) = 49.6 in. > 48.76 in.
Therefore d = 49.6 in. is used in further calculations.

V_d = Shear force due to total dead load at the critical section
= 96.07 kips

V_i = Factored shear force at the section due to externally applied loads occurring simultaneously with maximum moment, M_{max}
= $V_{mu} - V_d$

V_{mu} = Factored shear force occurring simultaneously with factored moment M_u, conservatively taken as design shear force at the section, V_u = 247.8 kips

V_i = 247.8 – 96.07 = 151.73 kips

M_{max} = Maximum factored moment at the critical section due to externally applied loads
= $M_u - M_d$

M_d = Bending moment at the critical section due to unfactored dead load = 254.36 k-ft. (see Table A.1.5.1)

M_{LL+I} = Bending moment at the critical section due to live load including impact = 146.19 k-ft. (see Table A.1.5.2)

M_u = Factored bending moment at the section
= $1.3(M_d + 1.67M_{LL+I})$
= 1.3[254.36 + 1.67(146.19)] = 648.05 k-ft.

M_{max} = 648.05 – 254.36 = 393.69 k-ft.

M_{cr} = Moment causing flexural cracking at the section due to externally applied loads

$$= \frac{I}{Y_t}(6\sqrt{f'_c} + f_{pe} - f_d)$$ [STD Eq. 9-28]

f_{pe} = Compressive stress in concrete due to effective prestress at the extreme fiber of the section where tensile stress is caused by externally applied loads which is the bottom fiber of the girder in the present case

$$= \frac{P_{se}}{A} + \frac{P_{se}e_x}{S_b}$$

P_{se} = Effective final prestress = 1,158.06 kips

$$f_{pe} = \frac{1,158.06}{788.4} + \frac{1,158.06(11.51)}{10,521.33} = 1.469 + 1.267 = 2.736 \text{ ksi}$$

f_d = Stress due to unfactored dead load at extreme fiber of the section where tensile stress is caused by externally applied loads which is the bottom fiber of the girder in the present case

$$= \left[\frac{M_g + M_S}{S_b} + \frac{M_{SDL}}{S_{bc}}\right]$$

M_g = Moment due to self-weight of the girder at the critical section = 112.39 k-ft. (see Table A.1.5.1)

M_S = Moment due to slab weight at the critical section = 109.52 k-ft. (see Table A.1.5.1)

M_{SDL} = Moment due to superimposed dead loads at the critical section = 32.45 k-ft.

S_b = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.^{3}

S_{bc} = Section modulus of the composite section referenced to the extreme bottom fiber of the precast girder
= 16,535.71 in.^{3}

$$f_d = \left[\frac{(112.39+109.52)(12\text{ in./ft.})}{10{,}521.33}+\frac{32.45(12\text{ in./ft.})}{16{,}535.71}\right]$$
$$= 0.253 + 0.024 = 0.277 \text{ ksi}$$

I = Moment of inertia about the centroid of the cross-section = 657,658.4 in^4

Y_t = Distance from centroidal axis of composite section to the extreme fiber in tension, which is the bottom fiber of the girder in the present case = 39.77 in.

$$M_{cr} = \frac{657{,}658.4}{39.772}\left(\frac{6\sqrt{5{,}582.5}}{1{,}000}+2.736-0.277\right)$$
$$= 48{,}074.23 \text{ k-in.} = 4{,}006.19 \text{ k-ft.}$$

$$V_{ci} = \frac{0.6\sqrt{5{,}582.5}}{1{,}000}(8)(49.6)+96.07+\frac{151.73(4{,}006.19)}{393.69}$$
$$= 17.79 + 96.07 + 1{,}544.00 = 1{,}657.86 \text{ kips}$$

Minimum $V_{ci} = 1.7\sqrt{f'_c}\,b'd$ [STD Art. 9.20.2.2]

$$= \frac{1.7\sqrt{5582.5}}{1000}(8)(49.6)$$
$$= 50.40 \text{ kips} << V_{ci} = 1{,}657.86 \text{ kips} \quad \text{(O.K.)}$$

Computation of V_{cw}: [STD Art. 9.20.2.3]

$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc})\, b'd + V_p$ [STD Eq. 9-29]

where:

V_{cw} = Nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, kips

f_{pc} = Compressive stress in concrete at centroid of cross-section resisting externally applied loads, ksi

$$= \frac{P_{se}}{A} - \frac{P_{se}e_x(y_{bcomp} - y_b)}{I} + \frac{M_D(y_{bcomp} - y_b)}{I}$$

P_{se} = Effective final prestress = 1,158.06 kips

e_x = Eccentricity of prestressing strands at the critical section for shear = 11.51 in.

y_{bcomp} = Lesser of y_{bc} and y_w, in.

y_{bc} = Distance from centroid of the composite section to the extreme bottom fiber of the precast girder = 39.77 in.

y_w = Distance from bottom fiber of the girder to the junction of the web and top flange
$= h - t_f - t_{fil}$

h = Depth of precast girder = 54 in.

t_f = Thickness of girder flange = 8 in.

t_{fil} = Thickness of girder fillets = 6 in.

y_w = 54 – 8 – 6 = 40 in. > y_{bc} = 39.77 in.

Therefore y_{bcomp} = 39.77 in.

y_b = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

M_D = Moment due to unfactored non-composite dead loads at the critical section
= 112.39 + 109.52 = 221.91 k-ft. (see Table A.1.5.1)

$$f_{pc} = \frac{1{,}158.06}{788.4} - \frac{1{,}158.06(11.51)(39.772 - 24.75)}{260{,}403} + \frac{221.91(12\text{ in./ft.})(39.772 - 24.75)}{260{,}403}$$

$= 1.469 - 0.769 + 0.154 = 0.854$ ksi

b' = Width of the web of a flanged member = 8 in.

d = Distance from the extreme compressive fiber to centroid of pretensioned reinforcement = 49.6 in.

V_p = Vertical component of prestress force for harped strands, kips
$= P_{se} \sin\Psi$

P_{se} = Effective prestress force for the harped strands, kips
= (number of harped strands)(area of strand)(effective final prestress)
= 10(0.153)(151.38) = 231.61 kips

Ψ = Angle of harped tendons to the horizontal, radians

$$= \tan^{-1}\left(\frac{h - y_{ht} - y_{hb}}{0.5(HD_e)}\right)$$

y_{ht} = Distance of the centroid of the harped strands from top fiber of the girder at girder end = 6 in. (see Fig. A.1.7.3)

y_{hb} = Distance of the centroid of the web strands from bottom fiber of the girder at hold down point = 6 in. (see Figure A.1.7.3)

HD_e = Distance of hold down point from the girder end
= 49.404 ft. (see Figure A.1.7.3)

$$\Psi = \tan^{-1}\left(\frac{54 - 6 - 6}{49.404\,(12\text{ in./ft.})}\right) = 0.071 \text{ radians}$$

$$V_p = 231.61 \sin(0.071) = 16.43 \text{ kips}$$

$$V_{cw} = \left(\frac{3.5\sqrt{5,582.5}}{1,000} + 0.3(0.854)\right)(8)(49.6) + 16.43 = 221.86 \text{ kips}$$

The allowable nominal shear strength provided by concrete, V_c is lesser of V_{ci} = 1,657.86 kips and V_{cw} = 221.86 kips

Therefore V_c = 221.86 kips

Shear reinforcement is not required if $2V_u \leq \phi V_c$

[STD Art. 9.20]

where:

V_u = Factored shear force at the section considered (calculated using load combination causing maximum shear force)
= 247.8 kips

ϕ = Strength reduction factor for shear = 0.90 for prestressed concrete members [STD Art. 9.14]

V_c = Nominal shear strength provided by concrete = 221.86 kips

$2\ V_u = 2(247.8) = 495.6$ kips $> \phi V_c = 0.9(221.86) = 199.67$ kips

Therefore shear reinforcement is required. The required shear reinforcement is calculated using the following criterion

$$V_u < \phi(V_c + V_s) \qquad \text{[STD Eq. 9-26]}$$

where V_s is the nominal shear strength provided by web reinforcement, kips

$$\text{Required } V_s = \frac{V_u}{\phi} - V_c = \frac{247.8}{0.9} - 221.86 = 53.47 \text{ kips}$$

Maximum shear force that can be carried by reinforcement

$$V_{s\,max} = 8\sqrt{f'_c}\,b'd \qquad \text{[STD Art. 9.20.3.1]}$$

where:

f'_c = Compressive strength of girder concrete at service
= 5,582.5 psi

$$V_{s\,max} = \frac{8\sqrt{5{,}582.5}}{1{,}000}(8)(49.6)$$

= 237.18 kips > Required V_s = 53.47 kips (OK)

The section depth is adequate for shear.

The required area of shear reinforcement is calculated using the following formula [STD Art. 9.20.3.1]

$$V_s = \frac{A_v f_y d}{s} \text{ or } \frac{A_v}{s} = \frac{V_s}{f_y d} \qquad \text{[STD Eq. 9-30]}$$

where:

A_v = Area of web reinforcement, in.^{2}

s = Center to center spacing of the web reinforcement, in.

f_y = Yield strength of web reinforcement = 60 ksi

$$\text{Required } \frac{A_v}{s} = \frac{(53.47)}{(60)(49.6)} = 0.018 \text{ in}^2\text{./in.}$$

Minimum shear reinforcement [STD Art. 9.20.3.3]

$$A_{v-min} = \frac{50\,b's}{f_y} \text{ or } \frac{A_{v\text{-}min}}{s} = \frac{50\,b'}{f_y} \qquad \text{[STD Eq. 9-31]}$$

$$\frac{A_{v\text{-}min}}{s} = \frac{(50)(8)}{60{,}000} = 0.0067 \text{ in.}^2\text{/in.} < \text{Required } \frac{A_v}{s} = 0.018 \text{ in}^2\text{./in.}$$

Therefore provide $\frac{A_v}{s} = 0.018$ in.^{2}/in.

Typically TxDOT uses double legged #4 Grade 60 stirrups for shear reinforcement. The same is used in this design.

A_v = Area of web reinforcement, in.^{2} = (number of legs)(area of bar) = 2(0.20) = 0.40 in.^{2}

Center-to-center spacing of web reinforcement

$$s = \frac{A_v}{\text{Required } \frac{A_v}{s}} = \frac{0.40}{0.018} = 22.22 \text{ in. say 22 in.}$$

$$V_s \text{ provided} = \frac{A_v f_y d}{s} = \frac{(0.40)(60)(49.6)}{22} = 54.1 \text{ kips}$$

Maximum spacing of web reinforcement is specified to be the lesser of 0.75 h_c or 24 in., unless V_s exceeds $4\sqrt{f'_c}\,b'd$.

[STD Art. 9.20.3.2]

$$4\sqrt{f'_c}\,b'd = \frac{4\sqrt{5{,}582.5}}{1{,}000}(8)(49.6)$$

$$= 118.59 \text{ kips} < V_s = 54.1 \text{ kips} \quad \text{(O.K.)}$$

Since V_s is less than the limit, maximum spacing of web reinforcement is given as

s_{max} = Lesser of 0.75 h_c or 24 in.

where:

h_c = Overall depth of the section = 62 in. (Note that the wearing surface thickness can also be included in the overall section depth calculations for shear. In the present case the wearing surface thickness of 1.5 in. includes the future wearing surface thickness and the actual wearing surface thickness is not specified. Therefore the wearing surface thickness is not included. This will not have any effect on the design)

$s_{max} = 0.75(62) = 46.5$ in. > 24 in.

Therefore maximum spacing of web reinforcement is $s_{max} = 24$ in.

Spacing provided, $s = 22$ in. < $s_{max} = 24$ in. (O.K.)

Therefore use # 4, double legged stirrups at 22 in. center-to-center spacing at the critical section.

The calculations presented above provide the shear design at the critical section. Different suitable sections along the span can be designed for shear using the same approach.

A.1.12 HORIZONTAL SHEAR DESIGN

[STD Art. 9.20.4]

The composite flexural members are required to be designed to fully transfer the horizontal shear forces at the contact surfaces of interconnected elements.

The critical section for horizontal shear is at a distance of $h_c/2$ (where h_c is the depth of composite section = 62 in.) from the face of the support. However, as the dimensions of the support are unknown in the present case, the critical section for shear is conservatively calculated from the centerline of the bearing support.

Distance of critical section for horizontal shear from bearing centerline:

$$h_c/2 = \frac{62\text{ in.}}{2(12\text{ in/ft.})} = 2.583\text{ ft.}$$

The cross-sections subject to horizontal shear shall be designed such that:

$$V_u \leq \phi V_{nh} \qquad \text{[STD Eq. 9-31a]}$$

where:

V_u = Factored shear force at the section considered (calculated using load combination causing maximum shear force) = 247.8 kips

V_{nh} = Nominal horizontal shear strength of the section, kips

ϕ = Strength reduction factor for shear = 0.90 for prestressed concrete members [STD Art. 9.14]

$$\text{Required } V_{nh} \geq \frac{V_u}{\phi} = \frac{247.8}{0.9} = 275.33\text{ kips}$$

The nominal horizontal shear strength of the section, V_{nh}, is determined based on one of the following applicable cases.

Case (a): When the contact surface is clean, free of laitance and intentionally roughened; the allowable shear force in pounds is given as:

$$V_{nh} = 80\, b_v\, d \qquad \text{[STD Art. 9.20.4.3]}$$

where:

b_v = Width of cross-section at the contact surface being investigated for horizontal shear = 20 in. (top flange width of the precast girder)

d = Distance from the extreme compressive fiber to centroid of pretensioned reinforcement
$= h_c - (y_b - e_x)$ [STD Art. 9.20.2.2]

h_c = Depth of the composite section = 62 in.

y_b = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

e_x = Eccentricity of prestressing strands at the critical section = 11.51 in.

d = 62 – (24.75 – 11.51) = 48.76 in.

$$V_{nh} = \frac{80(20)(48.76)}{1,000}$$
$= 78.02$ kips < Required $V_{nh} = 275.33$ kips (N.G.)

Case (b): When minimum ties are provided and contact surface is clean, free of laitance but not intentionally roughened; the allowable shear force in pounds is given as:

$$V_{nh} = 80\, b_v\, d \qquad \text{[STD Art. 9.20.4.3]}$$

$$V_{nh} = \frac{80(20)(48.76)}{1000}$$
$= 78.02$ kips < Required $V_{nh} = 275.33$ kips (N.G.)

Case (c): When minimum ties are provided and contact surface is clean, free of laitance and intentionally roughened to a full amplitude of approximately ¼ in.; the allowable shear force in pounds is given as:

$$V_{nh} = 350\, b_v\, d \qquad \text{[STD Art. 9.20.4.3]}$$

$$V_{nh} = \frac{350(20)(48.76)}{1{,}000}$$

$$= 341.32 \text{ kips} > \text{Required } V_{nh} = 275.33 \text{ kips} \quad \text{(O.K.)}$$

Design of ties for horizontal shear [STD Art. 9.20.4.5]

Minimum area of ties between the interconnected elements

$$A_{vh} = \frac{50\, b_v s}{f_y}$$

where:

A_{vh} = Area of horizontal shear reinforcement, in.^{2}

s = Center-to-center spacing of the web reinforcement taken as 22 in. This is the center to center spacing of web reinforcement which can be extended into the slab.

f_y = Yield strength of web reinforcement = 60 ksi

$A_{vh} = \frac{50(20)(22)}{60{,}000} = 0.37$ in.$^2 \approx 0.40$ in.^{2} (area of web reinforcement provided)

Maximum spacing of ties shall be:
s = Lesser of 4(least web width) and 24 in. [STD Art. 9.20.4.5.a]

Least web width = 8 in.

s = 4(8 in.) = 32 in. > 24 in. Therefore, use maximum s = 24 in.

Maximum spacing of ties = 24 in. which is greater than the provided spacing of ties = 22 in. (O.K.)

Therefore the provided web reinforcement shall be extended into the cast–in-place (CIP) slab to satisfy the horizontal shear requirements.

A.1.13 PRETENSIONED ANCHORAGE ZONE

[STD Art. 9.22]

A.1.13.1 Minimum Vertical Reinforcement

In a pretensioned girder, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4% of the total pretensioning force must be placed within the distance of $d/4$ of the girder end.

[STD Art. 9.22.1]

Minimum vertical stirrups at the each end of the girder:

P_s = Prestressing force before initial losses have occurred, kips
= (number of strands)(area of strand)(initial prestress)

Initial prestress, $f_{si} = 0.75\, f_s'$ [STD Art. 9.15.1]

where f_s' = Ultimate strength of prestressing strands = 270 ksi

$f_{si} = 0.75(270) = 202.5$ ksi

$P_s = 50(0.153)(202.5) = 1{,}549.13$ kips

Force to be resisted, F_s = 4% of $P_s = 0.04(1{,}549.13) = 61.97$ kips

Required area of stirrups to resist F_s

$$A_v = \frac{F_s}{\text{Unit Stress in stirrups}}$$

Unit stress in stirrups = 20 ksi

$$A_v = \frac{61.97}{20} = 3.1 \text{ in.}^2$$

Distance available for placing the required area of stirrups = $d/4$

where d is the distance from the extreme compressive fiber to centroid of pretensioned reinforcement = 48.76 in.

$$\frac{d}{4} = \frac{48.76}{4} = 12.19 \text{ in.}$$

Using 6 pairs of #5 bars @ 2 in. center to center spacing (within 12 in. from girder end) at each end of the girder

A_v = 2(area of each bar)(number of bars)
$= 2(0.31)(6) = 3.72 \text{ in.}^2 > 3.1 \text{ in.}^2$ (O.K.)

Therefore provide 6 pairs of #5 bars @ 2 in. center-to-center spacing at each girder end.

A.1.13.2 Confinement Reinforcement

STD Art. 9.22.2 specifies that the nominal reinforcement must be placed to enclose the prestressing steel in the bottom flange for a distance d from the end of the girder. [STD Art. 9.22.2]

where

d = Distance from the extreme compressive fiber to centroid of pretensioned reinforcement
= $h_c - (y_b - e_x) = 62 - (24.75 - 11.51) = 48.76$ in.

A.1.14 CAMBER AND DEFLECTIONS

A.1.14.1 Maximum Camber

The Standard Specifications do not provide any guidelines for the determination camber of prestressed concrete members. The Hyperbolic Functions Method proposed by Rauf and Furr (1970) for the calculation of maximum camber is used by TxDOT's prestressed concrete bridge design software, PSTRS14 (TxDOT 2004). The following steps illustrate the Hyperbolic Functions method for the estimation of maximum camber.

Step 1: The total prestressing force after initial prestress loss due to elastic shortening has occurred

$$P = \frac{P_i}{\left(1 + pn + \dfrac{e_c^2 A_s n}{I}\right)} + \frac{M_D\, e_c\, A_s\, n}{I\left(1 + pn + \dfrac{e_c^2 A_s n}{I}\right)}$$

where:

P_i = Anchor force in prestressing steel
= (number of strands)(area of strand)(f_{si})

f_{si} = Initial prestress before release = $0.75\, f_s'$
[STD Art. 9.15.1]

f_s' = Ultimate strength of prestressing strands = 270 ksi

f_{si} = 0.75(270) = 202.5 ksi

P_i = 50(0.153)(202.5) = 1549.13 kips

I = Moment of inertia of the non-composite precast girder
= 260403 in.^{4}

e_c = Eccentricity of prestressing strands at the midspan
= 19.47 in.

M_D = Moment due to self-weight of the girder at midspan
= 1209.98 k-ft.

A_s = Area of prestressing steel
= (number of strands)(area of strand)
= 50(0.153) = 7.65 in.^{2}

p = A_s/A

A = Area of girder cross-section = 788.4 in.^{2}

$$p = \frac{7.65}{788.4} = 0.0097$$

n = Modular ratio between prestressing steel and the girder concrete at release = E_s/E_{ci}

E_{ci} = Modulus of elasticity of the girder concrete at release
= $33(w_c)^{3/2}\sqrt{f'_{ci}}$ [STD Eq. 9-8]

w_c = Unit weight of concrete = 150 pcf

f'_{ci} = Compressive strength of precast girder concrete at release = 5,455 psi

$$E_{ci} = [33(150)^{3/2}\sqrt{5,455}\,]\left(\frac{1}{1,000}\right) = 4,477.63 \text{ ksi}$$

E_s = Modulus of elasticity of prestressing strands
= 28,000 ksi

n = 28,000/4,477.63 = 6.25

$$\left(1 + pn + \frac{e_c^2 A_s n}{I}\right) = 1 + (0.0097)(6.25) + \frac{(19.47^2)(7.65)(6.25)}{260,403}$$
$$= 1.130$$

$$P = \frac{1,549.13}{1.130} + \frac{(1,209.98)(12 \text{ in./ft.})(19.47)(7.65)(6.25)}{260,403(1.130)}$$

$$= 1370.91 + 45.93 = 1416.84 \text{ kips}$$

Initial prestress loss is defined as

$$PL_i = \frac{P_i - P}{P} = \frac{1549.13 - 1416.84}{1549.13} = 0.0854 = 8.54\%$$

Note that the values obtained for initial prestress loss and effective initial prestress force using this methodology are comparable with the values obtained in Section A.1.7.3.5. The effective prestressing force after initial losses, was found to be 1410.58 kips (comparable to 1416.84 kips) and the initial prestress loss was determined as 8.94% (comparable to 8.54%).

The stress in the concrete at the level of the centroid of the prestressing steel immediately after transfer is determined as follows.

$$f_{ci}^{s} = P\left(\frac{1}{A} + \frac{e_c^2}{I}\right) - f_c^s$$

where:

f_c^s = Concrete stress at the level of centroid of prestressing steel due to dead loads, ksi

$$= \frac{M_D\, e_c}{I} = \frac{(1{,}209.98)(12 \text{ in./ft.})(19.47)}{260{,}403} = 1.0856 \text{ ksi}$$

$$f_{ci}^{s} = 1416.84\left(\frac{1}{788.4} + \frac{19.47^2}{260{,}403}\right) - 1.0856 = 2.774 \text{ ksi}$$

The ultimate time dependent prestress loss is dependent on the ultimate creep and shrinkage strains. As the creep strains vary with the concrete stress, the following steps are used to evaluate the concrete stresses and adjust the strains to arrive at the ultimate prestress loss. It is assumed that the creep strain is proportional to the concrete stress and the shrinkage stress is independent of concrete stress. (Sinno 1970)

Step 2: Initial estimate of total strain at steel level assuming constant sustained stress immediately after transfer

$$\varepsilon_{c1}^{s} = \varepsilon_{cr}^{\infty} f_{ci}^{s} + \varepsilon_{sh}^{\infty}$$

where:

$\varepsilon_{cr}^{\infty}$ = Ultimate unit creep strain = 0.00034 in./in. [this value is prescribed by Sinno et. al. (1970)]

$\varepsilon_{sh}^{\infty}$ = Ultimate unit shrinkage strain = 0.000175 in./in. [this value is prescribed by Sinno et. al. (1970)]

$\varepsilon_{c1}^{s} = 0.00034(2.774) + 0.000175 = 0.001118$ in./in.

Step 3: The total strain obtained in Step 2 is adjusted by subtracting the elastic strain rebound as follows

$$\varepsilon_{c2}^{s} = \varepsilon_{c1}^{s} - \varepsilon_{c1}^{s} E_s \frac{A_s}{E_{ci}}\left(\frac{1}{A} + \frac{e_c^2}{I}\right)$$

$$\varepsilon_{c2}^{s} = 0.001118 - 0.001118(28{,}000)\frac{7.65}{4{,}477.63}\left(\frac{1}{788.4} + \frac{19.47^2}{260{,}403}\right)$$

$= 0.000972$ in./in.

Step 4: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_c^{s} = \varepsilon_{c2}^{s} E_s A_s \left(\frac{1}{A} + \frac{e_c^2}{I}\right)$$

$$\Delta f_c^{s} = 0.000972(28{,}000)(7.65)\left(\frac{1}{788.4} + \frac{19.47^2}{260{,}403}\right) = 0.567 \text{ ksi}$$

Step 5: The total strain computed in Step 2 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\varepsilon_{c4}^{s} = \varepsilon_{cr}^{\infty}\left(f_{ci}^{s} - \frac{\Delta f_c^{s}}{2}\right) + \varepsilon_{sh}^{\infty}$$

$$\varepsilon_{c4}^{s} = 0.00034\left(2.774 - \frac{0.567}{2}\right) + 0.000175 = 0.00102 \text{ in./in.}$$

Step 6: The total strain obtained in Step 5 is adjusted by subtracting the elastic strain rebound as follows

$$\varepsilon_{c5}^{s} = \varepsilon_{c4}^{s} - \varepsilon_{c4}^{s} E_s \frac{A_s}{E_{ci}}\left(\frac{1}{A} + \frac{e_c^2}{I}\right)$$

$$\varepsilon_{c5}^{s} = 0.00102 - 0.00102(28000)\ \frac{7.65}{4477.63}\left(\frac{1}{788.4} + \frac{19.47^2}{260403}\right)$$

$= 0.000887$ in./in

Sinno (1970) recommends stopping the updating of stresses and adjustment process after Step 6. However, as the difference between the strains obtained in Steps 3 and 6 is not negligible, this process is carried on until the total strain value converges.

Step 7: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_{c1}^{s} = \varepsilon_{c5}^{s} E_s A_s \left(\frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\Delta f_{c1}^{s} = 0.000887(28{,}000)(7.65)\left(\frac{1}{788.4} + \frac{19.47^2}{260{,}403} \right) = 0.5176 \text{ ksi}$$

Step 8: The total strain computed in Step 5 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\varepsilon_{c6}^{s} = \varepsilon_{cr}^{\infty} \left(f_{ci}^{s} - \frac{\Delta f_{c1}^{s}}{2} \right) + \varepsilon_{sh}^{\infty}$$

$$\varepsilon_{c6}^{s} = 0.00034\left(2.774 - \frac{0.5176}{2} \right) + 0.000175 = 0.00103 \text{ in./in.}$$

Step 9: The total strain obtained in Step 8 is adjusted by subtracting the elastic strain rebound as follows

$$\varepsilon_{c7}^{s} = \varepsilon_{c6}^{s} - \varepsilon_{c6}^{s} E_s \frac{A_s}{E_{ci}} \left(\frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\varepsilon_{c7}^{s} = 0.00103 - 0.00103(28{,}000)\ \frac{7.65}{4{,}477.63}\left(\frac{1}{788.4} + \frac{19.47^2}{260{,}403} \right)$$

$$= 0.000896 \text{ in./in}$$

The strains have sufficiently converged and no more adjustments are needed.

Step 10: Computation of final prestress loss

Time dependent loss in prestress due to creep and shrinkage strains is given as

$$PL^{\infty} = \frac{\varepsilon_{c7}^{s} E_s A_s}{P_i} = \frac{0.000896(28{,}000)(7.65)}{1{,}549.13} = 0.124 = 12.4\%$$

Total final prestress loss is the sum of initial prestress loss and the time dependent prestress loss expressed as follows

$$PL = PL_i + PL^{\infty}$$

where:

PL = Total final prestress loss percent.

PL_i = Initial prestress loss percent = 8.54%

PL^{∞}= Time dependent prestress loss percent = 12.4%

PL = 8.54 + 12.4 = 20.94% (This value of final prestress loss is less than the one estimated in Section A.1.7.3.6. where the final prestress loss was estimated to be 25.24%)

Step 11: The initial deflection of the girder under self-weight is calculated using the elastic analysis as follows:

$$C_{DL} = \frac{5\,w\,L^4}{384 E_{ci} I}$$

where:

C_{DL} = Initial deflection of the girder under self-weight, ft.

w = Self-weight of the girder = 0.821 kips/ft.

L = Total girder length = 109.67 ft.

E_{ci} = Modulus of elasticity of the girder concrete at release = 4,477.63 ksi = 644,778.72 k/ft.^{2}

I = Moment of inertia of the non-composite precast girder = 260403 in.^{4} = 12.558 ft.^{4}

$$C_{DL} = \frac{5(0.821)(109.67^4)}{384(644{,}778.72)(12.558)} = 0.191 \text{ ft.} = 2.29 \text{ in.}$$

Step 12: Initial camber due to prestress is calculated using the moment area method. The following expression is obtained from the M/EI diagram to compute the camber resulting from the initial prestress.

$$C_{pi} = \frac{M_{pi}}{E_{ci} I}$$

where:

M_{pi} = $[0.5(P)(e_e)(0.5L)^2 + 0.5(P)(e_c - e_e)(0.67)(HD)^2 + 0.5P(e_c - e_e)(HD_{dis})(0.5L + HD)]/(Eci)(I)$

P = Total prestressing force after initial prestress loss due to elastic shortening have occurred = 1,416.84 kips

HD = Hold down distance from girder end = 49.404 ft. = 592.85 in. (see Figure A.1.7.3)

HD_{dis} = Hold down distance from the center of the girder span = 0.5(109.67) – 49.404 = 5.431 ft. = 65.17 in.

e_e = Eccentricity of prestressing strands at girder end = 11.07 in.

e_c = Eccentricity of prestressing strands at midspan = 19.47 in.

L = Overall girder length = 109.67 ft. = 1,316.04 in.

$$M_{pi} = \{0.5(1{,}416.84)(11.07)[(0.5)(1{,}316.04)]^2 + 0.5(1{,}416.84)(19.47 - 11.07)(0.67)(592.85)^2 + 0.5(1{,}416.84)(19.47 - 11.07)(65.17)[0.5(1316.04) + 592.85]\}$$

$$M_{pi} = 3.396 \text{ x } 10^9 + 1.401 \text{ x } 10^9 + 0.485 \text{ x } 10^9 = 5.282 \text{ x } 10^9$$

$$C_{pi} = \frac{5.282 \times 10^9}{(4{,}477.63)(260{,}403)} = 4.53 \text{ in.} = 0.378 \text{ ft.}$$

Step 13: The initial camber, C_I, is the difference between the upward camber due to initial prestressing and the downward deflection due to self-weight of the girder.

$$C_i = C_{pi} - C_{DL} = 4.53 - 2.29 = 2.24 \text{ in.} = 0.187 \text{ ft.}$$

Step 14: The ultimate time-dependent camber is evaluated using the following expression.

$$\text{Ultimate camber } C_t = C_i\,(1 - PL^{\infty})\frac{\varepsilon_{cr}^{\infty}\left(f_{ci}^{s} - \dfrac{\Delta f_{c1}^{s}}{2}\right) + \varepsilon_e^{s}}{\varepsilon_e^{s}}$$

where:

$$\varepsilon_e^{s} = \frac{f_{ci}^{s}}{E_{ci}} = \frac{2.774}{4477.63} = 0.000619 \text{ in./in.}$$

$$C_t = 2.24(1 - 0.124)\frac{0.00034\left(2.774 - \dfrac{0.5176}{2}\right) + 0.000619}{0.000619}$$

$C_t = 4.673$ in. = 0.389 ft. ↑

A.1.14.2 Deflection Due to Slab Weight

The deflection due to the slab weight is calculated using an elastic analysis as follows.

Deflection of the girder at midspan

$$\Delta_{slab1} = \frac{5\, w_s\, L^4}{384\, E_c\, I}$$

where:

w_s = Weight of the slab = 0.80 kips/ft.

E_c = Modulus of elasticity of girder concrete at service

$= 33(w_c)^{3/2}\sqrt{f_c'}$

$= 33(150)^{1.5}\sqrt{5{,}582.5}\left(\dfrac{1}{1{,}000}\right) = 4{,}529.66$ ksi

I = Moment of inertia of the non-composite girder section
= 260,403 in.^{4}

L = Design span length of girder (center to center bearing)
= 108.583 ft.

$$\Delta_{slab1} = \frac{5\left(0.80/12 \text{ in./ft.}\right)[(108.583)(12 \text{ in./ft.})]^4}{384(4{,}529.66)(260{,}403)}$$

= 2.12 in. = 0.177 ft. ↓

Deflection at quarter span due to slab weight

$$\Delta_{slab2} = \frac{57\, w_s\, L^4}{6144\, E_c\, I}$$

$$\Delta_{slab2} = \frac{57\left(0.80\big/_{12\text{ in./ft.}}\right)[(108.583)(12\text{ in./ft.})]^4}{6{,}144(4{,}529.66)(260{,}403)}$$

$= 1.511$ in. $= 0.126$ ft. ↓

A.1.14.3 Deflections Due to Superimposed Dead Loads

Deflection due to barrier weight at midspan

$$\Delta_{barr1} = \frac{5\, w_{barr}\, L^4}{384\, E_c\, I_c}$$

where:

w_{barr} = Weight of the barrier = 0.109 kips/ft.

I_c = Moment of inertia of composite section = 657,658.4 in^4

$$\Delta_{barr1} = \frac{5\left(0.109\big/_{12\text{ in./ft.}}\right)[(108.583)(12\text{ in./ft.})]^4}{384(4{,}529.66)(657{,}658.4)}$$

$= 0.114$ in. $= 0.0095$ ft. ↓

Deflection at quarter span due to barrier weight

$$\Delta_{barr2} = \frac{57\, w_{barr}\, L^4}{6144\, E_c\, I}$$

$$\Delta_{barr2} = \frac{57\left(0.109\big/_{12\text{ in./ft.}}\right)[(108.583)(12\text{ in./ft.})]^4}{6{,}144(4{,}529.66)(657{,}658.4)}$$

$= 0.0815$ in. $= 0.0068$ ft. ↓

Deflection due to wearing surface weight at midspan

$$\Delta_{ws1} = \frac{5\, w_{ws}\, L^4}{384\, E_c\, I_c}$$

where

w_{ws} = Weight of wearing surface = 0.128 kips/ft.

$$\Delta_{ws1} = \frac{5\left(0.128/12\text{ in./ft.}\right)[(108.583)(12\text{ in./ft.})]^4}{384(4,529.66)(657,658.4)}$$

$$= 0.134\text{ in.} = 0.011\text{ ft.} \downarrow$$

Deflection at quarter span due to wearing surface

$$\Delta_{ws2} = \frac{57\, w_{ws}\, L^4}{6144\, E_c\, I}$$

$$\Delta_{ws2} = \frac{57\left(0.128/12\text{ in./ft.}\right)[(108.583)(12\text{ in./ft.})]^4}{6,144(4,529.66)(657,658.4)}$$

$$= 0.096\text{ in.} = 0.008\text{ ft.} \downarrow$$

A.1.14.4 Total Deflection Due to Dead Loads

The total deflection at midspan due to slab weight and superimposed loads is:

$$\Delta_{T1} = \Delta_{slab1} + \Delta_{barr1} + \Delta_{ws1}$$

$$= 0.177 + 0.0095 + 0.011 = 0.1975\text{ ft.} \downarrow$$

The total deflection at quarter span due to slab weight and superimposed loads is:

$$\Delta_{T2} = \Delta_{slab2} + \Delta_{barr2} + \Delta_{ws2}$$

$$= 0.126 + 0.0068 + 0.008 = 0.1408\text{ ft.} \downarrow$$

The deflections due to live loads are not calculated in this example as they are not a design factor for TxDOT bridges.

A.1.15 COMPARISON OF RESULTS FROM DETAILED DESIGN AND PSTRS14

The prestressed concrete bridge girder design program, PSTRS14 (TxDOT 2004), is used by TxDOT for bridge design. The PSTRS14 program was run with same parameters as used in this detailed design and the results of the detailed example and PSTRS14 program are compared in table A.1.15.1.

Table A.1.15.1. Comparison of the Results from PSTRS14 Program with Detailed Design Example

Parameter		PSTRS 14 Result	Detailed Design Result	Percent Difference
Live Load Distribution Factor		0.727	0.727	0.00
Initial Prestress Loss		8.93%	8.94%	-0.11
Final Prestress Loss		25.23%	25.24%	-0.04
Girder Stresses at Transfer				
At Girder End	Top Fiber	35 psi	35 psi	0.00
	Bottom Fiber	3,274 psi	3,273 psi	0.03
At Transfer Length Section	Top Fiber	Not Calculated	104 psi	-
	Bottom Fiber	Not calculated	3,215 psi	-
At Hold Down	Top Fiber	319 psi	351 psi	-10.03
	Bottom Fiber	3,034 psi	3,005 psi	1.00
At Midspan	Top Fiber	335 psi	368 psi	-9.85
	Bottom Fiber	3,020 psi	2,991 psi	0.96
Girder Stresses at Service				
At Girder End	Top Fiber	29 psi	Not Calculated	-
	Bottom Fiber	2,688 psi	Not Calculated	-
At Midspan	Top Fiber	2,563 psi	2,562 psi	0.04
	Bottom Fiber	-414 psi	-412 psi	0.48
Slab Top Fiber Stress		Not Calculated	658 psi	-
Required Concrete Strength at Transfer		5,457 psi	5,455 psi	0.04
Required Concrete Strength at Service		5,585 psi	5,582.5 psi	0.04
Total Number of Strands		50	50	0.00
Number of Harped Strands		10	10	0.00
Ultimate Flexural Moment Required		6,771 k-ft.	6,769.37 k-ft.	0.02
Ultimate Moment Provided		8,805 k-ft	8,936.56 k-ft.	-1.50
Shear Stirrup Spacing at the Critical Section: double legged #4 bars		21.4 in.	22 in.	-2.80
Maximum Camber		0.306 ft.	0.389 ft.	-27.12
Deflections				
Slab Weight	Midspan	-0.1601 ft.	0.1770 ft.	-11.00
	Quarter Span	-0.1141 ft.	0.1260 ft.	-10.00
Barrier Weight	Midspan	-0.0096 ft.	0.0095 ft.	1.04
	Quarter Span	-0.0069 ft.	0.0068 ft.	1.45
Wearing Surface Weight	Midspan	-0.0082 ft.	0.0110 ft.	-34.10
	Quarter Span	-0.0058 ft.	0.0080 ft.	-37.60

Except for a few differences, the results from the detailed design are in good agreement with the PSTRS 14 (TxDOT 2004) results. The causes for the differences in the results are discussed as follows.

1. *Girder stresses at transfer*: The detailed design example uses the overall girder length of 109'-8" for evaluating the stresses at transfer at the midspan section and hold down point locations. The PSTRS 14 uses the design span length of 108'-7" for this calculation. This causes a difference in the stresses at transfer at hold down point locations and midspan. The use of full girder length for stress calculations at transfer conditions seems to be appropriate as the girder rests on the ground and the resulting moment is due to the self-weight of the overall girder.

2. *Maximum Camber*: The difference in the maximum camber results from detailed design and PSTRS 14 (TxDOT 2001) is occurring due to two reasons.

    a. The detailed design example uses the overall girder length for the calculation of initial camber whereas, the PSTRS 14 program uses the design span length.

    b. The updated composite section properties, based on the modular ratio between slab and actual girder concrete strengths are used for the camber calculations in the detailed design. However, PSTRS 14 program does not update the composite section properties.

3. *Deflections*: The difference in the deflections is occurring due to the use of updated section properties and elastic modulus of concrete in the detailed design, based on the optimized concrete strength. However, PSTRS 14 program does not update the composite section properties and uses the elastic modulus of concrete based on the initial input.

Appendix B

Detailed Examples for Interior Texas U54 Prestressed Concrete Bridge Girder Design

DRAFT
August 29, 2005

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B.2 Interior Texas U54 Prestressed Concrete Bridge Girder Design Using AASHTO LRFD Specifications

B.2.1 *INTRODUCTION*

Following is a detailed design example showing sample calculations for design of a typical Interior Texas prestressed precast concrete U54 beam supporting a single span bridge. The design is based on the *AASHTO LRFD Bridge Design Specifications, U.S., 3rd Edition 2004*. The recommendations provided by the TxDOT Bridge Design Manual (TxDOT 2001) are considered in the design. The number of strands and concrete strength at release and at service are optimized using the TxDOT methodology.

B.2.2 *DESIGN PARAMETERS*

The bridge considered for design has a span length of 110 ft. (c/c abutment distance), a total width of 46 ft. and total roadway width of 44 ft. The bridge superstructure consists of four Texas U54 beams spaced 11.5 ft. center-to-center designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck as shown in Figure B.2.1. The wearing surface thickness is 1.5 in., which includes the thickness of any future wearing surface. T501 type rails are considered in the design. AASHTO LRFD HL93 is the design live load. The relative humidity (RH) of 60% is considered in the design. The bridge cross-section is shown in Figure B.2.1.

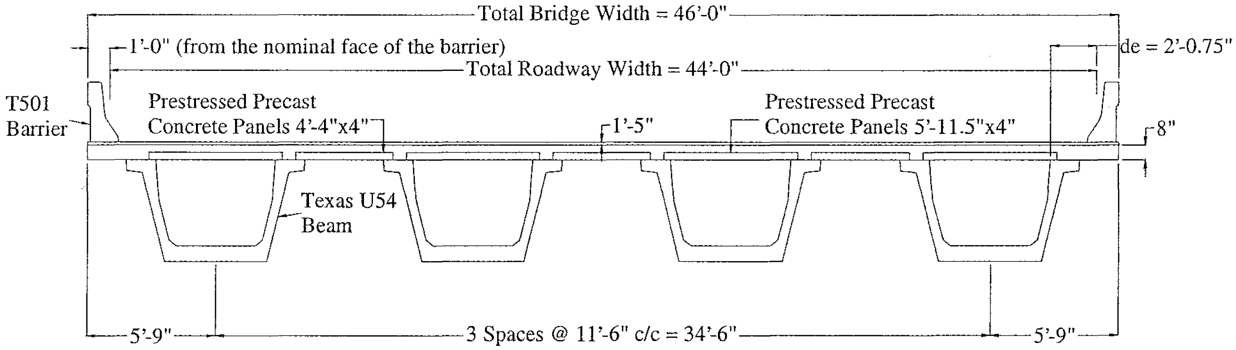

Figure B.2.1 Bridge Cross-Section Details

B.2.3 *MATERIAL PROPERTIES*

Cast-in-place slab:

Thickness t_s = 8.0 in.

Concrete Strength at 28-days, f'_c = 4,000 psi

Unit weight of concrete = 150 pcf

Wearing surface:

Thickness of asphalt wearing surface (including any future wearing surfaces), t_w = 1.5 in.

Unit weight of asphalt wearing surface = 140 pcf

Precast beams: Texas U54 beam

Concrete Strength at release, f'_{ci} = 4,000 psi*

Concrete strength at 28 days, $f_c' = 5{,}000$ psi*

Concrete unit weight = 150 pcf

*This value is taken as initial estimate and will be updated based on most optimum design

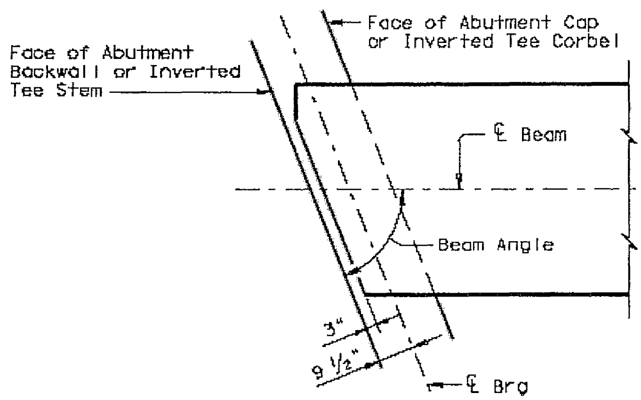

Figure B.2.2 Beam End Detail for Texas U54 Beams (TxDOT 2001)

From Figure B.2.2.

Span length (c/c Piers) = 110 ft. – 0 in.

Overall beam length = 110 ft. – 2(3 in.) = 109 ft. – 6 in.

Design span = 110 ft. – 2(9.5 in.) = 108 ft. – 5 in.

= 108.417 ft. (c/c of bearing)

Prestressing strands: ½ in. diameter, seven wire low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate tensile strength, f_{pu} = 270,000 psi [LRFD Table 5.4.4.1-1]

Yield strength, $f_{py} = 0.9\ f_{pu}$ = 243,000 psi [LRFD Table 5.4.4.1-1]

Modulus of elasticity, E_s = 28,500 ksi [LRFD Art. 5.4.4.2]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

before transfer, $f_{pi} \leq 0.75\ f_{pu}$ = 202,500 psi

at service limit state(after all losses) $f_{pe} \leq 0.80\ f_{py}$ = 194,400 psi

Non-prestressed reinforcement:

Yield strength, f_y = 60,000 psi

Modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Traffic barrier:

T501 type barrier weight = 326 plf /side

B.2.4 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

B.2.4.1 Non-Composite Section

The section properties of a Texas U54 girder as described in the TxDOT Bridge Design Manual (TxDOT 2001) are provided in Table B.2.1. The strand pattern and section geometry are shown in Figures B.2.3 and B.2.4.

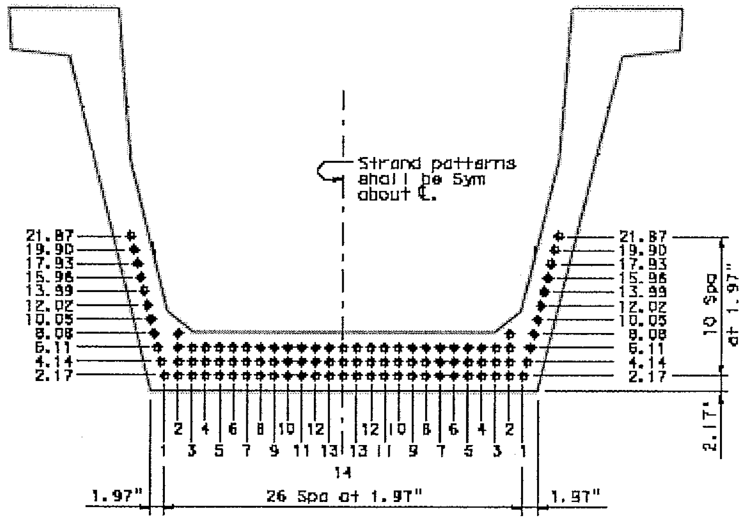

Figure B.2.3 Strand Pattern for Texas U54 Beams (TxDOT 2001)

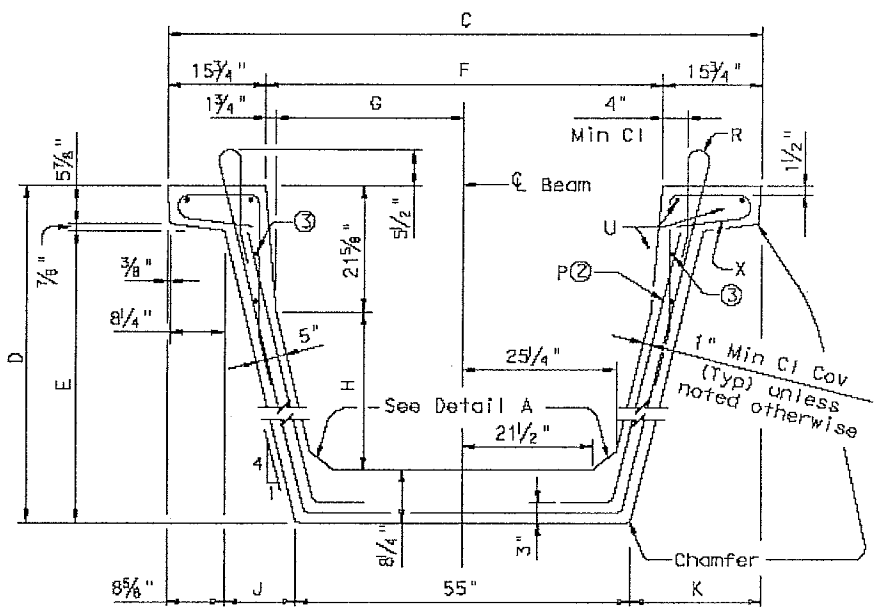

Figure B.2.4 Typical Section of Texas U54 Beams (TxDOT 2001)

Table B.2.1 Section Properties of Texas U54 beams (notations as used in Figure B.2.4, Adapted from TxDOT Bridge Design Manual (TxDOT 2001))

C	D	E	F	G	H	J	K	y_t	y_b	Area	I	Weight
in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in^2.	in^4.	plf
96	54	47.25	64.5	30.5	24.125	11.875	20.5	31.58	22.36	1,120	403,020	1,167

where,

I = moment of inertia about the centroid of the non-composite precast beam

y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam

y_t = distance from centroid to the extreme top fiber of the non-composite precast beam

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/y_b = 403,020/22.36 = 18,024.15 in.^{3}

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/y_t = 403,020/31.58 = 12,761.88 in.^{3}

B.2.4.2 Composite Section

B.2.4.2.1 Effective Flange Width

According to the LRFD Specifications, C4.6.2.6.1, the effective flange width of the U54 beam is determined as though each web is an individual supporting element.

The effective flange width of each web may be taken as the least of

[LRFD Art. 4.6.2.6.1]

- 1/4×(effective girder span length): $\dfrac{108.417 \text{ ft. } (12 \text{ in./ft.})}{4} = 325.25$ in.
- 12×(Average depth of slab) + greater of (web thickness or one-half the width of the top flange of the girder (web, in this case))

  = 12× (8.0 in.) + greater of (5 in. or 15.75 in./2) = 103.875 in.
- The average spacing of the adjacent beams (webs, in this case)

  = 69 in. = 5.75 ft. (controls)

  For the entire U-beam the effective flange width is 2×(5.75 ft. ×12) = 138 in.

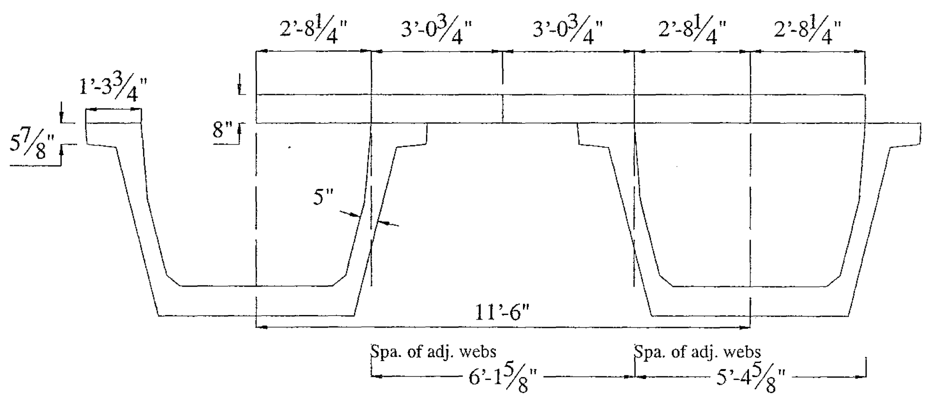

Figure B.2.5 Effective Flange Width Calculations

B.2.4.2.2 Modular Ratio Between Slab and Beam Material

Following the TxDOT Bridge Design Manual (TxDOT 2001) recommendation the modular ratio between the slab and girder concrete is taken as 1. This assumption is used for service load design calculations. For the flexural strength limit design, shear design, and deflection calculations, the actual modular ratio based on optimized concrete strengths is used.

$$n = \left(\frac{E_c \text{ for slab}}{E_c \text{ for beam}}\right) = 1$$

B.2.4.2.3 Transformed Section Properties

Transformed flange width = n× (effective flange width) = 1(138 in.) = 138 in.

Transformed Flange Area = n× (effective flange width) (t_s) = 1(138 in.)(8 in.)
= 1,104 in.^{2}

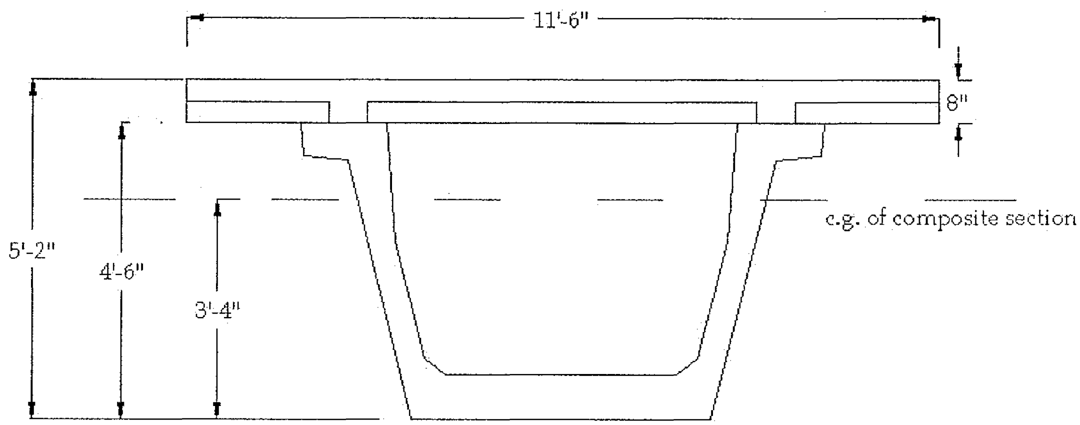

Figure B.2.6 Composite Section

Table B.2.2 Properties of Composite Section

	Transformed Area in.^{2}	y_b in.	$A\,y_b$ in.	$A(y_{bc} - y_b)^2$	I in.^{4}	$I + A(y_{bc} - y_b)^2$ in.^{4}
Beam	1,120	22.36	25,043.2	350,488.43	403,020	753,508.43
Slab	1,104	58	64,032	355,711.62	5,888	361,599.56
∑	2,224		89,075.2			1,115,107.99

A_c = total area of composite section = 2,224 in.^{2}

h_c = total height of composite section = 62 in.

I_c = moment of inertia of composite section = 1,115,107.99 in.^{4}

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam = 89,075.2 / 2,224 = 40.05 in.

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam = 54 – 40.05 = 13.95 in.

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab = 62 – 40.05 = 21.95 in.

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam = I_c/y_{bc} = 1,115,107.99 / 40.05 = 27,842.9 in.^{3}

S_{tg} = composite section modulus for top fiber of the precast beam = I_c/y_{tg} = 1,115,107.99 / 13.95 = 79,936.06 in.^{3}

S_{tc} = composite section modulus for top fiber of the slab = I_c/y_{tc} = 1,115,107.99 / 21.95 = 50,802.19 in.^{3}

B.2.5 SHEAR FORCES AND BENDING MOMENTS

B.2.5.1 Shear Forces and Bending Moments Due to Dead Loads

B.2.5.1.1 Dead Loads

Self-weight of the beam = 1.167 kips/ft. [TxDOT Bridge Design Manual]

Weight of CIP deck and precast panels on each beam

$$= (0.150 \text{ pcf})\left(\frac{8 \text{ in.}}{12 \text{ in./ft.}}\right)\left(\frac{138 \text{ in.}}{12 \text{ in./ft.}}\right)$$

$$= 1.15 \text{ kips/ft.}$$

B.2.5.1.2 Superimposed Dead Loads

B.2.5.1.2.1 Due to Diaphragm

TxDOT Bridge Design Manual (TxDOT 2001) requires two interior diaphragms with U54 beam, located as close as 10 ft. from the midspan of the beam. Shear forces and bending moment values in the interior beam can be calculated by the following equations:

For x = 0 ft. – 44.21 ft.

V_x = 3 kips $\qquad$ $M_x = 3x$ kips

For x = 44.21 ft. – 54.21 ft.

V_x = 0 kips $\qquad$ $M_x = 3x - 3(x - 44.21)$ kips

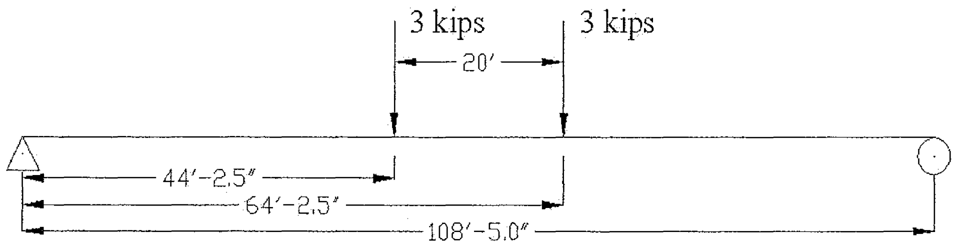

Figure B.2.7 Location of interior diaphragms on a simply supported bridge girder.

For U54 beam bridge design, TxDOT accounts for haunches in designs that require special geometry and where the haunch will be large enough to have a significant impact on the overall beam. Since this study is for typical bridges, a haunch will not be included for U54 beams for composite properties of the section and additional dead load considerations.

B.2.5.1.2.2 Due to T501 Rail

TxDOT Bridge Design Manual recommends (TxDOT 2001, Chap. 7 Sec. 24) that 1/3 of the rail dead load should be used for an interior beam adjacent to the exterior beam.

$$\text{Weight of T501 rails or barriers on each interior beam} = \left(\frac{326 \text{ plf}/1000}{3}\right)$$

$$= 0.109 \text{ kips/ft./interior beam}$$

B.2.5.1.2.3 Due to Wearing Surface

$$\text{Weight of 1.5 in. wearing surface} = \frac{(0.140 \text{ pcf})\left(\frac{1.5 \text{ in.}}{12 \text{ in./ft.}}\right)(44 \text{ ft.})}{4 \text{ beams}} = 0.193 \text{ kips/ft.}$$

Total superimposed dead load = 0.109 + 0.193 = 0.302 kips/ft.

The LRFD specifications, Art. 4.6.2.2.1, states that permanent loads (rail, sidewalks and future wearing surface) may be distributed uniformly among all beams if the following conditions are met:

Width of the deck is constant O.K.
Number of beams, N_b, is not less than four (N_b = 4) O.K.
The roadway part of the overhang, $d_e \leq 3.0$ ft.
$d_e = 5.75 - 1.0 - 55/(2\times12) - 4.75/12 = 2.063$ ft. O.K.
Curvature in plan is less than 4° (curvature is 0.0) O.K.
Cross-section of the bridge is consistent with one of the cross-sections given in Table 4.6.2.2.1-1 in LRFD Specifications O.K.

Since these criteria are satisfied, the wearing surface loads are equally distributed among the 4 beams.

B.2.5.1.3 Unfactored Shear Forces and Bending Moments

Shear forces and bending moments in the beam due to dead loads, superimposed dead loads at every tenth of the design span, and at critical sections (midspan and critical section for shear) are provided in this section. The critical section for shear design is determined by an iterative procedure later in the example. The bending moment (M) and shear force (V) due to uniform dead loads and uniform superimposed dead loads at any section at a distance x are calculated using the following formulae, where the uniform dead load is denoted as w.

$$M = 0.5wx\,(L - x)$$

$$V = w\,(0.5L - x)$$

The shear forces and bending moments due to dead loads and superimposed dead loads are shown in Tables B.2.3 and B.2.4.

Table B.2.3 Shear Forces due to Dead loads

Distance	Section	Non-Composite Dead Loads				Superimposed Dead Loads		Total Dead Load Shear Force
x	x/L	Beam Wt. V_g	Slab Wt. V_{slab}	Diaphram Wt. V_{dia}	Total Wt. $V_g+V_{slab}+V_{dia}$	Barrier Wt. V_b	Wearing Surface Wt. V_{ws}	
ft.		kips	kips	kips	kips	kips	kips	kips
0.375	0.003	62.82	61.91	3.00	127.73	5.87	10.39	143.99
5.503	0.051	56.84	56.01	3.00	115.85	5.31	9.40	130.56
10.842	0.100	50.61	49.87	3.00	103.48	4.73	8.37	116.58
21.683	0.200	37.96	37.40	3.00	78.36	3.55	6.28	88.19
32.525	0.300	25.30	24.94	3.00	53.24	2.36	4.18	59.78
43.367	0.400	12.65	12.47	3.00	28.12	1.18	2.09	31.39
54.209	0.500	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table B.2.4 Bending Moment due to Dead loads

Distance	Section	Non-Composite Dead Loads				Superimposed Dead Loads		Total Dead Load Bending Moment
x	x/L	Beam Wt. M_g	Slab Wt. M_{slab}	Diaphram Wt. M_{dia}	Total Wt. $M_{slab} + M_{dia}$	Barrier Wt. M_b	Wearing Surface Wt. M_{ws}	
ft.		k – ft.	k – ft.	k – ft.	k – ft.	k – ft.	k – ft.	k – ft.
0.375	0.003	23.64	23.30	1.13	24.43	2.21	3.91	54.19
5.503	0.051	330.46	325.64	16.51	342.15	30.87	54.65	758.13
10.842	0.100	617.29	608.30	32.53	640.83	57.66	102.09	1,417.87
21.683	0.200	1,097.36	1,081.38	65.05	1,146.43	102.50	181.48	2,527.77
32.525	0.300	1,440.30	1,419.32	97.58	1,516.90	134.53	238.20	3,329.93
43.367	0.400	1,646.07	1,622.09	130.10	1,752.19	153.75	272.23	3,824.24
54.209	0.500	1,714.65	1,689.67	132.63	1,822.30	160.15	283.57	3,980.67

B.2.5.2
Shear Forces and Bending Moments due to Live Load

B.2.5.2.1
Live Load

Design live load is HL93, which consists of a combination of: [LRFD Art. 3.6.1.2.1]

1. Design truck or design tandem with dynamic allowance [LRFD Art. 3.6.1.2.2] [LRFD Art. 3.6.1.2.3]
2. Design lane load of 0.64 kips/ft. without dynamic allowance [LRFD Art. 3.6.1.2.4]

B1.5.2.2
Live Load Distribution Factor for Typical Interior Beam

The live load bending moments and shear forces are determined by using the simplified distribution factor formulas, [LRFD Art. 4.6.2.2]. To use the simplified live load distribution factor formulas, the following conditions are met: [LRFD Art. 4.6.2.2]

Width of the slab is constant O.K.
Number of beams, N_b, is not less than four ($N_b = 4$) O.K.
Beams are parallel and of the same stiffness O.K.
The roadway part of the overhang, $d_e \leq 3.0$ ft.

$d_e = 5.75 - 1.0 - 55/(2\times 12) - 4.75/12 = 2.063$ ft. O.K.

Curvature in plan is less than 4° (curvature is 0.0) O.K.
Cross-section of the bridge is consistent with one of the cross-sections given in [LRFD Table 4.6.2.2.1-1], the bridge type is (c) O.K

The number of design lanes is computed as:

Number of design lanes = the integer part of the ratio of (w/12), where (w) is the clear roadway width, in ft., between curbs/or barriers [LRFD Art. 3.6.1.1.1]

$w = 44$ ft.

Number of design lanes = integer part of (44 ft./12) = 3 lanes

B.2.5.2.3
Distribution Factor for Bending Moment

For all limit states except fatigue limit state:

For two or more design lanes loaded:

$$DFM = \left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125}$$ [LRFD Table 4.6.2.2.2b-1]

Provided that:	$6.0 \leq S \leq 18.0$;	$S = 11.5$ ft.	O.K.
	$20 \leq L \leq 140$;	$L = 108.417$ ft.	O.K.
	$18 \leq d \leq 65$;	$d = 54$ in.	O.K.
	$N_b \geq 3$;	$N_b = 4$	O.K.

where,

DFM = live load moment distribution factor for interior beam

S = beam spacing, ft.

L = beam span, ft.

d = depth of the beam, ft.

N_b = Number of beam

$$DFM = \left(\frac{11.5}{6.3}\right)^{0.6}\left(\frac{11.5\times 54}{12.0\times(108.417)^2}\right)^{0.125} = 0.728 \text{ lanes/beam}$$

For one design lane loaded:

$$DFM = \left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12.0L^2}\right)^{0.25}$$ [LRFD Table 4.6.2.2.2b-1]

$$DFM = \left(\frac{11.5}{3.0}\right)^{0.35}\left(\frac{11.5\times 54}{12.0\times(108.417)^2}\right)^{0.25} = 0.412 \text{ lanes/beam}$$

Thus, the case for two or more lanes loaded controls and DFM = 0.728 lanes/beam.

B.2.5.2.4 Distribution Factor for Fatigue

- For fatigue limit state:

The LRFD Specifications, Art.3.4.1, states that for fatigue limit state, a single design truck should be used. However, live load distribution factors given in Art. 4.6.2.2, LRFD Specifications, take into consideration the multiple presence factor, *m*. Art.3.6.1.1.2 states that the multiple presence factor, *m*, for one design lane loaded is 1.2. Therefore, the distribution factor for one design lane loaded with the multiple presence factor removed, should be used. The distribution factor for fatigue limit state is:

= 0.412/1.2 = 0.344 lanes/beam.

B.2.5.2.5 Distribution Factor for Shear Force

For two or more design lanes loaded:

$$DFV = \left(\frac{S}{7.4}\right)^{0.8}\left(\frac{d}{12.0L}\right)^{0.1}$$ [LRFD Table 4.6.2.2.3a-1]

Provided that:	$6.0 \leq S \leq 18.0$;	S = 11.5 ft.	O.K.
	$20 \leq L \leq 140$;	L = 110 ft.	O.K.
	$18 \leq d \leq 65$;	d = 54 in.	O.K.
	$N_b \geq 3$;	$N_b = 4$	O.K.

where,

DFV = live load shear distribution factor for interior beam

S = beam spacing, ft.

L = beam span, ft.

d = depth of the beam, ft.

N_b = number of beam

$$DFV = \left(\frac{11.5}{7.4}\right)^{0.8}\left(\frac{54}{12.0\times 108.417}\right)^{0.1} = 1.035 \text{ lanes/beam}$$

For one design lane loaded:

$$DFV = \left(\frac{S}{10}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1}$$ [LRFD Table 4.6.2.2.3a-1]

$$DFV = \left(\frac{11.5}{10}\right)^{0.6}\left(\frac{54}{12.0\times 108.417}\right)^{0.1} = 0.791 \text{ lanes/beam}$$

Thus, the case for two or more lanes loaded controls and DFV = 1.035 lanes/beam

B.2.5.2.6 Dynamic Allowance

IM = 33%

[LRFD Table 3.6.2.1.-1]

where, IM = dynamic load allowance, applied to truck load only

B.2.5.2.7 Unfactored Shear Forces and Bending Moments

B.2.5.2.7.1 Due to Truck Load, V_{LT} and M_{LT}

For all limit states except for fatigue limit state:

Shear force and bending moment envelopes on a per-lane-basis due to HL93 truck loadings are calculated at tenth-points of the span using the following equations given in PCI Bridge Design Manual (PCI 2003):

For x/L = 0 – 0.333

Maximum unfactored bending moment, $M = \dfrac{72(x)[(L - x) - 9.33]}{L}$

For x/L = 0.333 – 0.5

Maximum unfactored bending moment, $M = \dfrac{72(x)[(L - x) - 4.67]}{L} - 112$

For x/L = 0 – 0.5

Maximum unfactored shear force, $V = \dfrac{72[(L - x) - 9.33]}{L}$

V_{LT} = (Unfactored shear force per lane) (DFV) (1+IM)
= (Unfactored shear force per lane) (1.035) (1+0.33)
= (Unfactored shear force per lane) (1.378) kips

M_{LT} = (Unfactored bending moment per lane) (DFM) (1+IM)
= (Unfactored bending moment per lane) (0.728) (1+0.33)
= (Unfactored bending moment per lane) (0.968) k-ft.

B.2.5.2.7.2 Due to Tandem Load, V_{TA} and M_{TA}

Shear force and bending moment envelopes on a per-lane-basis due to HL93 tandem loadings are calculated at tenth-points of the span using the following equations:

For $x/L = 0 – 0.5$

Maximum unfactored bending moment, $M = 50(x)\left(\dfrac{L-x-2}{L}\right)$

For $x/L = 0 – 0.5$

Maximum unfactored shear force, $V = 50\left(\dfrac{L-x-2}{L}\right)$

The factored bending moment and shear forces are calculated in the same way as for the HL93 truck loading, as shown above.

_B.2.5.2.7.3 M_f Due to Fatigue Truck Load,_

For fatigue limit state:
The fatigue load is a single design truck which has the same axle weight used in all other limit states but with a constant spacing of 30.0 ft. between the 32.0 kip axles. Bending moment envelope on a per-lane-basis is calculated using the equations given in the PCI Bridge Design Manual (PCI 2003):

For $x/L = 0 – 0.241$

Maximum unfactored bending moment, $M = \dfrac{72(x)[(L - x) - 18.22]}{L}$

For $x/L = 0.241 – 0.5$

Maximum unfactored bending moment, $M = \dfrac{72(x)[(L - x) - 11.78]}{L} - 112$

M_f = (Unfactored bending moment per lane) (DFM) (1+IM)
= (Unfactored bending moment per lane) (0.344) (1+0.15)
= (Unfactored bending moment per lane) (0.395) k-ft.

B.2.5.2.7.4
Due to Lane Load, V_{LL} and M_{LL}

The bending moments and shear forces due to uniformly distributed lane load of 0.64 kip/ft. are calculated using the following equations given in PCI Bridge Design Manual (PCI 2003):

Maximum unfactored bending moment, $M_x = 0.5(w)(x)(L - x)$

Maximum unfactored shear force, $V_x = \dfrac{0.32 \times (L - x)^2}{L}$ $for \quad x \leq 0.5L$

where,

x = distance from the support to the section at which bending moment or shear force is calculated

L = span length = 108.417 ft.

w = uniform load per linear foot of load lane = 0.64 klf

where, V_x is in kips/lane and M_x is in k-ft./lane

Lane load shear force and bending moment per typical interior beam are as follows:

V_{LL} = (Unfactored shear force per lane) (DFV)
= (Unfactored shear force per lane) (1.035) kips

M_{LL} = (Unfactored bending moment per lane) (DFM)
= (Unfactored bending moment per lane) (0.728) k-ft.

B.2.5.3
Limit States: Load Combinations

Total factored load shall be taken as [LRFD Eq. 3.4.1-1]

$Q = \eta \sum \gamma_i q_i$

where,

[LRFD Table 1.3.2]

η = a factor relating to ductility, redundancy and operational importance (Here, η is considered to be 1.0)

γ_i = load factors

[LRFD Table 3.4.1-1]

q_i = specified loads

Service I: Check compressive stresses in prestressed concrete components:
$Q = 1.00(DC + DW) + 1.00(LL + IM)$ [LRFD Table 3.4.1-1]

Service III: Check tensile stresses in prestressed concrete components:
$Q = 1.00(DC + DW) + 0.80(LL + IM)$ [LRFD Table 3.4.1-1]

Strength I: Check ultimate strength: [LRFD Table 3.4.1-1 & 2]
Maximum $Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)$

Minimum $Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)$

Fatigue: Check stress range in strands

$Q = 0.75(LL + IM)$ [LRFD Table. 3.4.1-1]

Table B.2.5 Shear forces and Bending moments due to Live loads

Distance	Section	Truck Load with impact (controls)		Lane Load		Tandem Load with impact		Fatigue Truck with Impact
		V_{LT}	M_{LT}	V_{LL}	M_{LL}	V_{TA}	M_{TA}	M_f
x	x/L	Shear	Moment	Shear	Moment	Shear	Moment	Moment
ft.		kips	k-ft.	kips	k-ft.	kips	k-ft.	k-ft.
0.375	0.000	90.24	23.81	35.66	9.44	67.32	17.76	8.84
6.000	0.055	85.10	359.14	32.04	143.15	64.06	247.97	115.07
10.842	0.100	80.67	615.45	29.08	246.55	60.67	462.71	225.69
21.683	0.200	70.76	1,079.64	22.98	438.30	53.79	820.41	389.70
32.525	0.300	60.85	1,392.64	17.59	575.27	46.91	1,073.17	502.76
43.370	0.400	50.93	1,575.96	12.93	657.47	40.03	1,220.96	561.76
54.210	0.500	41.03	1,618.96	8.98	684.85	33.14	1,263.76	559.09

B.2.6 ESTIMATION OF REQUIRED PRESTRESS

B.2.6.1 Service Load Stresses at Midspan

The preliminary estimate of the required prestress and number of strands is based on the stresses at midspan

Bottom tensile stresses (SERVICE III) at midspan due to applied loads

$$f_b = \frac{M_g + M_S}{S_b} + \frac{M_b + M_{ws} + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

Top compressive stresses (SERVICE I) at midspan due to applied loads

$$f_t = \frac{M_g + M_S}{S_t} + \frac{M_b + M_{ws} + M_{LT} + M_{LL}}{S_{tg}}$$

where,

f_b = concrete stress at the bottom fiber of the beam, ksi

f_t = concrete stress at the top fiber of the beam, ksi

M_g = Unfactored bending moment due to beam self-weight, k-ft.

M_S = Unfactored bending moment due to slab and diaphragm weight, k-ft.

M_b = Unfactored bending moment due to barrier weight, k-ft.

M_{ws} = Unfactored bending moment due to wearing surface, k-ft.

M_{LT} = Factored bending moment due to truck load, k-ft.

M_{LL} = Factored bending moment due to lane load, k-ft.

Substituting the bending moments and section modulus values, bottom tensile stress at midspan is:

$$f_b = \frac{(1,714.65+1,689.67+132.63)(12)}{18,024.15} + \frac{(160.15+283.57+0.8\times(1,618.3+684.57))(12)}{27,842.9}$$
$$= 3.34 \text{ ksi}$$

$$f_t = \frac{(1,714.65+1,689.67+132.63)(12)}{12,761.88} + \frac{(160.15+283.57+1,618.3+684.57)(12)}{79,936.06}$$
$$= 3.738 \text{ ksi}$$

B.2.6.2 Allowable Stress Limit

At service load conditions, allowable tensile stress is

f'_c = specified 28-day concrete strength of beam (initial guess), 5,000 psi

$F_b = 0.19\sqrt{f'_c(ksi)} = 0.19\sqrt{5} = 0.425$ ksi [LRFD Table. 5.9.4.2.2-1]

B.2.6.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – allowable tensile stress at final $= f_b - F_b$

$= 3.34 - 0.425 = 2.915$ ksi

Assuming the distance from the center of gravity of strands to the bottom fiber of the beam is equal to $y_{bs} = 2$ in.

Strand eccentricity at midspan:

$$e_c = y_b - y_{bs} = 22.36 - 2 = 20.36 \text{ in.}$$

Bottom fiber stress due to prestress after losses:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}\, e_c}{S_b}$$

where, P_{se} = effective pretension force after all losses

$$2.915 = \frac{P_{se}}{1120} + \frac{20.36\, P_{se}}{18024.15}$$

Solving for P_{se} we get,

$P_{se} = 1,441.319$ kips

Assuming final losses = 20% of f_{pi}

Assumed final losses = 0.2(202.5 ksi) = 40.5 ksi

The prestress force per strand after losses

= (cross-sectional area of one strand) (f_{pe})

= 0.153×(202.5-40.5) = 24.786 kips

Number of strands required = 1441.319 /24.786 = 58.151

Try 60 – ½ in. diameter, 270 ksi strands

Strand eccentricity at midspan after strand arrangement

$$e_c = 22.36 - \frac{27(2.17)+27(4.14)+6(6.11)}{60} = 18.91 \text{ in.}$$

P_{se} = 60(24.786) = 1,487.16 kips

$$f_b = \frac{1487.16}{1120} + \frac{18.91(1487.16)}{18024.15}$$

$$= 1.328 + 1.56 = 2.888 \text{ ksi} < 2.915 \text{ ksi} \qquad \text{(N.G.)}$$

Try 62 – ½ in. diameter, 270 ksi strands

Strand eccentricity at midspan after strand arrangement

$$e_c = 22.36 - \frac{27(2.17)+27(4.14)+8(6.11)}{62} = 18.824 \text{ in.}$$

P_{se} = 62(24.786) = 1,536.732 kips

$$f_b = \frac{1536.732}{1120} + \frac{18.824(1536.732)}{18024.15}$$

$$= 1.372 + 1.605 = 2.977 \text{ ksi} > 2.915 \text{ ksi}$$

Therefore, use 62 strands

Number of Strands	Distance from bottom (in.)
27	2.17
27	4.14
8	6.11

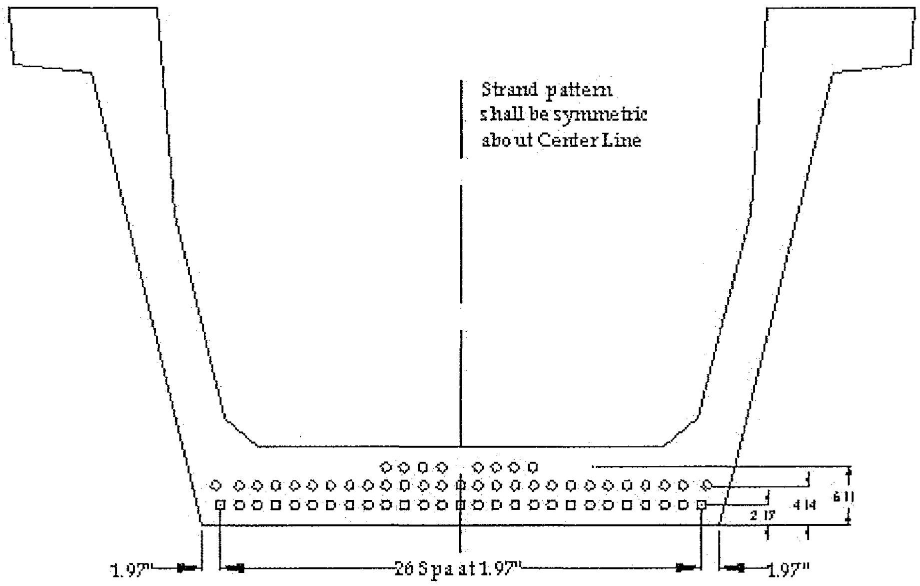

Fig. B.2.8 Initial Strand Pattern

B.2.7 PRESTRESS LOSSES

Total prestress losses = $\Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ [LRFD Eq. 5.9.5.1-1]

where,

Δf_{pSR} = loss of prestress due to concrete shrinkage

Δf_{pES} = loss of prestress due to elastic shortening

Δf_{pCR} = loss of prestress due to creep of concrete

Δf_{pR2} = loss of prestress due to relaxation of Prestressing steel after transfer

Number of strands = 62

A number of iterations will be performed to arrive at the optimum f_c' and f_{ci}'

B.2.7.1
Iteration 1
B.2.7.1.1
Concrete Shrinkage

$\Delta f_{pSR} = (17.0 - 0.15\ H)$ [LRFD Eq. 5.9.5.4.2-1]

where, H is the relative humidity = 60%

$$\Delta f_{pSR} = [17.0 - 0.150(60)]\frac{1}{1000} = 8 \text{ ksi}$$

B.2.7.1.2
Elastic Shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Eq. 5.9.5.2.3a-1]

where,

$$f_{cgp} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c^2}{I} - \frac{(M_g)e_c}{I}$$

The LRFD Specifications, Art. 5.9.5.2.3a, states that f_{cgp} can be calculated on the basis of prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, we will assume the initial losses as a percentage of initial prestressing stress before release, f_{pi}. The assumed initial losses shall be be checked and if different from the assumed value, a second iteration will be carried on. Moreover, iterations may also be required if the f'_{ci} value doesn't match that calculated in a previous step.

f_{cgp} = sum of the concrete stresses at the center of gravity of the prestressing tendons due to prestressing force and the self-weight of the member at the sections of the maximum moment (ksi)
P_{si} = pretension force after allowing for the initial losses,
As the initial losses are unknown at this point, 8% initial loss in prestress is assumed as a first estimate.
= (number of strands)(area of each strand)[0.92(0.75 f_{pu})]
= 62(0.153)(0.92)(0.75)(270) = 1,767.242 kips

M_g = Unfactored bending moment due to beam self-weight = 1714.64 k-ft.

e_c = eccentricity of the strand at the midspan = 18.824 in.

$$f_{cgp} = \frac{1767.242}{1120} + \frac{1767.242(18.824)^2}{403020} - \frac{1714.64(12)(18.824)}{403020}$$

$$= 1.578 + 1.554 - 0.961 = 2.171 \text{ ksi}$$

Initial estimate for concrete strength at release, f'_{ci} = 4000 psi

$$E_{ci} = (150)^{1.5}(33)\sqrt{4000}\ \frac{1}{1000} = 3834.254 \text{ ksi}$$

$$\Delta f_{pES} = \frac{28500}{3834.254}(2.171) = 16.137 \text{ ksi}$$

B.2.7.1.3
Creep of Concrete

$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

where,

Δf_{cdp} = change in the concrete stress at center of gravity of prestressing steel due to permanent loads, with the exception of the load acting at the time the prestressing force is applied. Values of Δf_{cdp} should be calculated at the same section or at sections for which f_{cgp} is calculated. (ksi)

$$\Delta f_{cdp} = \frac{(M_{slab} + M_{dia})e_c}{I} + \frac{(M_b + M_{ws})(y_{bc} - y_{bs})}{I_c}$$

where,

y_{bc} = 40.05 in.

y_{bs} = the distance from center of gravity of the strand at midspan to the bottom of the beam = 22.36 – 18.824 = 3.536 in.

I = moment of inertia of the non-composite section =403,020 in.^{4}

I_c = moment of inertia of composite section = 1,115,107.99 in.^{4}

$$f_{cdp} = \frac{(1689.67+132.63)(12)(18.824)}{403020} + \frac{(160.15+283.57)(12)(37.54 - 3.536)}{1115107.99}$$

$= 1.021 + 0.174 = 1.195$ ksi

$\Delta f_{pCR} = 12(2.171) - 7(1.195) = 17.687$ ksi.

B.2.7.1.4 Relaxation of Prestressing Steel

For pretensioned members with 270 ksi low-relaxation strand conforming to AASHTO M 203 [LRFD Art. 5.9.5.4.4c]

Relaxation loss after Transfer,

$\Delta f_{pR2} = 30\%[20.0 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$ [LRFD Eq. 5.9.5.4.4c-1]

$= 0.3[20.0 - 0.4(16.137) - 0.2(8 + 17.687)] = 2.522$ ksi

Relaxation loss before Transfer,

Initial relaxation loss, Δf_{pR1}, is generally determined and accounted for by the Fabricator. However, Δf_{pR1} is calculated and included in the losses calculations for demonstration purpose and alternatively, it can be assumed to be zero. A total of 0.5 day time period is assumed between stressing of strands and initial transfer of prestress force. As per LRFD Commentary C.5.9.5.4.4, f_{pj} is assumed to be $0.8 \times f_{pu}$ for this example.

$$\Delta f_{pR1} = \frac{\log(24.0 \times t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$ [LRFD Eq. 5.9.5.4.4b-2]

$$= \frac{\log(24.0 \times 0.5 \text{ day})}{40.0}\left[\frac{216}{243} - 0.55\right]216 = 1.975 \text{ ksi}$$

Δf_{pR1} will remain constant for all the iterations and Δf_{pR1} = 1.975 ksi will be used throughout the losses calculation procedure.

Total initial prestress loss = $\Delta f_{pES} + \Delta f_{pR1}$ = 16.137 + 1.975 = 18.663 ksi

$$\text{Initial Prestress loss} = \frac{(\Delta f_{ES} + \Delta f_{pR1}) \times 100}{0.75 f_{pu}} = \frac{[16.137+1.975]100}{0.75(270)}$$

= 8.944% > 8% (assumed initial prestress losses)

Therefore, next trial is required assuming 8.944% initial losses

Δf_{pES} = 8 ksi [LRFD Eq. 5.9.5.4.2-1]

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Eq. 5.9.5.2.3a-1]

where,

$$f_{cgp} = \frac{P_{si}}{A} + \frac{P_{si}\, e_c^2}{I} - \frac{(M_g)e_c}{I}$$

P_{si} = pretension force after allowing for the initial losses, assuming 8.944% initial losses = (number of strands)(area of each strand)[0.9106(0.75 f_{pu})]

= 62(0.153)(0.9106)(0.75)(270) = 1,749.185 kips

$$f_{cgp} = \frac{1749.185}{1120} + \frac{1749.185\,(18.824)^2}{403020} - \frac{1714.65(12)(18.824)}{403020}$$

= 1.562 + 1.538 – 0.961 = 2.139 ksi

Assuming f'_{ci} = 4,000 psi

$$E_{ci} = (150)^{1.5}(33)\sqrt{4000}\ \frac{1}{1000} = 3{,}834.254 \text{ ksi}$$

$$\Delta f_{pES} = \frac{28500}{3834.254}(2.139) = 15.899 \text{ ksi}$$

$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

Δf_{cdp} is same as calculated in the previous trial.

Δf_{cdp} = 1.195 ksi

$\Delta f_{pCR} = 12(2.139) - 7(1.195) = 17.303$ ksi.

For pretensioned members with 270 ksi low-relaxation strand conforming to AASHTO M 203 [LRFD Art. 5.9.5.4.4c]

$\Delta f_{pR2} = 30\%[20.0 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$

$= 0.3[20.0 - 0.4(15.899) - 0.2(8 + 17.303)] = 2.574$ ksi

Total initial prestress loss = $\Delta f_{pES} + \Delta f_{pR1} = 15.899 + 1.975 = 17.874$ ksi

$$\text{Initial Prestress loss} = \frac{(\Delta f_{ES} + \Delta f_{pR1}) \times 100}{0.75 f_{pu}} = \frac{[15.899 + 1.975]100}{0.75(270)}$$

$= 8.827\% < 8.944\%$ (assumed initial prestress losses)

Therefore, next trial is required assuming 8.827% initial losses

$\Delta f_{pES} = 8$ ksi [LRFD Eq. 5.9.5.4.2-1]

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Eq. 5.9.5.2.3a-1]

where,

$$f_{cgp} = \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g)e_c}{I}$$

P_{si} = pretension force after allowing for the initial losses, assuming 8.827% initial losses = (number of strands)(area of each strand)[0.9117(0.75 f_{pu})]

$= 62(0.153)(0.9117)(0.75)(270) = 1{,}751.298$ kips

$$f_{cgp} = \frac{1751.298}{1120} + \frac{1751.298(18.824)^2}{403020} - \frac{1714.65(12)(18.824)}{403020}$$

$= 1.564 + 1.54 - 0.961 = 2.143$ ksi

Assuming $f'_{ci} = 4{,}000$ psi

$$E_{ci} = (150)^{1.5}(33)\sqrt{4000}\ \frac{1}{1000} = 3{,}834.254 \text{ ksi}$$

$$\Delta f_{pES} = \frac{28500}{3834.254}(2.143) = 15.929 \text{ ksi}$$

$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

Δf_{cdp} is same as calculated in the previous trial.

Δf_{cdp} = 1.193 ksi
Δf_{pCR} = 12(2.143) – 7(1.193) = 17.351 ksi.

For pretensioned members with 270 ksi low-relaxation strand conforming to AASHTO M 203 [LRFD Art. 5.9.5.4.4c]

Δf_{pR2} = 30%[20.0 – 0.4 Δf_{pES} – 0.2(Δf_{pSR} + Δf_{pCR})

= 0.3[20.0 – 0.4(15.929) – 0.2(8 + 17.351)] = 2.567 ksi

Total initial prestress loss = Δf_{pES} + Δf_{pR1} = 15.929 + 1.975= 17.904 ksi

$$\text{Initial prestress loss} = \frac{(\Delta f_{ES} + \Delta f_{pR1}) \times 100}{0.75 f_{pu}} = \frac{[15.929+2.526]100}{0.75(270)}$$

= 8.841% ≈ 8.827% (assumed initial prestress losses)

B.2.7.1.5
Total Losses at Transfer

Total initial losses = Δf_{ES} = 15.929 + 1.975= 17.904 ksi

f_{si} = effective initial prestress = 202.5 – 17.904 = 184.596 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 62(0.153)(184.596) = 1,751.078 kips

B.2.7.1.6
Total Losses at Service Loads

Δf_{SR} = 8 ksi

Δf_{ES} = 15.929 ksi

Δf_{R2} = 2.567 ksi

Δf_{CR} = 17.351 ksi

Total final losses = 8 + 15.929 + 2.567 + 17.351 = 45.822 ksi

$$\text{or } \frac{45.822\ (100)}{0.75(270)} = 22.63\%$$

f_{se} = effective final prestress = 0.75(270) – 45.822 = 156.678 ksi

P_{se} = 62(0.153)(156.678) = 1,486.248 kips

B.2.7.1.7
Final Stresses at Midspan

Bottom fiber stress in concrete at midspan at service load

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\, e_c}{S_b} - f_b$$

$$f_{bf} = \frac{1486.248}{1120} + \frac{18.824(1486.248)}{18024.15} - 3.34 = 1.327 + 1.552 - 3.34$$

$= -0.461$ ksi $>$ -0.425 ksi (allowable) (N.G.)

This shows that 62 strands are not adequate. Therefore, try 64 strands

$$e_c = 22.36 - \frac{27(2.17)+27(4.14)+10(6.11)}{62} = 18.743 \text{ in}$$

$P_{se} = 64(0.153)(156.678) = 1534.191$ kips

$$f_{bf} = \frac{1534.191}{1120} + \frac{18.743(1534.191)}{18024.15} - 3.34 = 1.370 + 1.595 - 3.34$$

$= -0.375$ ksi $<$ -0.425 ksi (allowable) (O.K.)

Therefore, use 64 strands.

Allowable tension in concrete = $0.19\sqrt{f'_c(ksi)}$

$$f'_{c\ reqd.} = \left(\frac{0.375}{0.19}\right)^2 \times 1000 = 3{,}896 \text{ psi}$$

Top fiber stress in concrete at midspan at service loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1534.191}{1120} - \frac{18.743(1534.191)}{12761.88} + 3.737$$

$= 1.370 - 2.253 + 3.737 = 2.854$ ksi

Allowable compression stress limit for all load combinations = $0.6\ f'_c$

$f'_{c\ reqd} = 2854/0.6 = 4{,}757$ psi

Top fiber stress in concrete at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}$$

$$= \frac{1534.191}{1120} - \frac{18.743(1534.191)}{12761.88} + \frac{(1714.65 + 1689.67 + 132.63)(12)}{12761.88} + \frac{(160.15 + 283.57)(12)}{79936.06}$$

$= 1.370 - 2.253 + 3.326 + 0.067 = 2.510$ ksi

Allowable compression stress limit for effective pretension force + permanent dead loads = 0.45 f'_c

$f'_{c\ reqd.}$ = 2510/0.45 = 5,578 psi (controls)

Top fiber stress in concrete at midspan due to live load + ½(effective prestress + dead loads)

$$f_{tf} = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}\right)$$

$$= \frac{(1618.3+684.57)(12)}{79936.06} + 0.5\left(\frac{1534.191}{1120} - \frac{18.743(1534.191)}{12761.88} + \frac{(1714.65+1689.67+132.63)(12)}{12761.88} + \frac{(160.15+283.57)(12)}{79936.06}\right)$$

$$= 0.346 + 0.5(1.370 - 2.253 + 3.326 + 0.067) = 1.601 \text{ ksi}$$

Allowable compression stress limit for effective pretension force + permanent dead loads = 0.4 f'_c

$f'_{c\ \text{reqd.}}$ = 1601/0.4 = 4,003 psi

B.2.7.1.8 *Initial Stresses at End*

Since P_{si} = 64 (0.153) (184.596) = 1,807.564 kips

Initial concrete stress at top fiber of the beam at midspan

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\,e_c}{S_t} + \frac{M_g}{S_t}$$

where, M_g = moment due to beam self-weight at girder end = 0 k-ft.

$$f_{ti} = \frac{1807.564}{1120} - \frac{18.743(1807.564)}{12761.88} = 1.614 - 2.655 = -1.041 \text{ ksi}$$

Tension stress limit at transfer = $0.24\sqrt{f'_{ci}(ksi)}$

Therefore, $f'_{ci\ \text{reqd.}} = \left(\frac{1.041}{0.24}\right)^2 \times 1000$ = 18,814 psi

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1807.564}{1120} + \frac{18.743(1807.564)}{18024.15}$$

$$= 1.614 + 1.88 = 3.494 \text{ ksi}$$

Compression stress limit at transfer = 0.6 f'_{ci}

Therefore, $f'_{ci \text{ reqd.}} = \dfrac{3494}{0.6} = 5{,}823$ psi

B.2.7.1.9 Debonding of Strands and Debonding Length

The calculation for initial stresses at the girder end show that preliminary estimate of $f'_{ci} = 4{,}000$ psi is not adequate to keep the tensile and compressive stresses at transfer within allowable stress limits as per LRFD Art. 5.9.4.1. Therefore, debonding of strands is required to keep the stresses within allowable stress limits.

In order to be consistent with the TxDOT design procedures, the debonding of strands is carried out in accordance with the procedure followed in PSTRS14 (TxDOT 2004).

Two strands are debonded at a time at each section located at uniform increments of 3 ft. along the span length, beginning at the end of the girder. The debonding is started at the end of the girder because due to relatively higher initial stresses at the end, greater number of strands are required to be debonded, and debonding requirement, in terms of number of strands, reduces as the section moves away from the end of the girder. In order to make the most efficient use of debonding due to greater eccentricities in the lower rows, the debonding at each section begins at the bottom most row and goes up. Debonding at a particular section will continue until the initial stresses are within the allowable stress limits or until a debonding limit is reached. When the debonding limit is reached, the initial concrete strength is increased and the design cycles to convergence. As per TxDOT Bridge Design Manual (TxDOT 2001) and AASHTO LRFD Art. 5.11.4.3, the limits of debonding for partially debonded strands are described as follows:

1. Maximum percentage of debonded strands per row
    a. TxDOT Bridge Design Manual (TxDOT 2001) recommends a maximum percentage of debonded strands per row should not exceed 75%.

   b. AASHTO LRFD recommends a maximum percentage of debonded strands per row should not exceed 40%.
2. Maximum percentage of debonded strands per section
   a. TxDOT Bridge Design Manual (TxDOT 2001) recommends a maximum percentage of debonded strands per section should not exceed 75%.
   b. AASHTO LRFD recommends a maximum percentage of debonded strands per section should not exceed 25%.
3. LRFD requires that not more than 40% of the debonded strands or four strands, whichever is greater, shall have debonding terminated at any section.
4. Maximum length of debonding
   a. TxDOT Bridge Design Manual (TxDOT 2001) recommends to use the maximum debonding length chosen to be lesser of the following:
      i. 15 ft.
      ii. 0.2 times the span length, or
      iii. half the span length minus the maximum development length as specified in the 1996 AASHTO Standard Specifications for Highway Bridges, Section 9.28. However, for the purpose of demonstration, the maximum development length will be calculated as specified in AASHTO LRFD Art. 5.11.4.2 and Art. 5.11.4.3.
   b. AASHTO LRFD recommends, "the length of debonding of any strand shall be such that all limit states are satisfied with consideration of the total developed resistance at any section being investigated.
5. AASHTO LRFD further recommends, "debonded strands shall be symmetrically distributed about the center line of the member. Debonded lengths of pairs of strands that are

symmetrically positioned about the centerline of the member shall be equal. Exterior strands in each horizontal row shall be fully bonded."

The recommendations of TxDOT Bridge Design Manual regarding the debonding percentage per section per row and maximum debonding length as described above are followed in this detailed design example.

B.2.7.1.10 Maximum Debonding Length

As per TxDOT Bridge Design Manual (TxDOT 2001), the maximum debonding length is the lesser of the following:

a. 15 ft.

b. 0.2 (L), or

c. 0.5 (L) - l_d

where, l_d is the development length calculated based on AASHTO LRFD Art. 5.11.4.2 and Art. 5.11.4.3. as follows:

$$l_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \qquad \text{[LRFD Eq. 5.11.4.2-1]}$$

where,

l_d = development length (in.)

κ = 2.0 for pretensioned strands [LRFD Art. 5.11.4.3]

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

= 156.276 (ksi)

d_b = nominal strand diameter = 0.5 in.

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

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ [LRFD Eq. 5.7.3.1.1-1]

k = 0.28 for low-relaxation strand [LRFD Table C5.7.3.1.1-1]

For Rectangular Section Behavior,

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{0.85 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1-4]

$d_p = h - y_{bs}$ = 62 – 3.617= 58.383 in.

β_1 = 0.85 for f'c $f'_c \le 4.0$ ksi [LRFD Art. 5.7.2.2]

$= 0.85 - 0.05(f'_c - 4.0) \le 0.65$ for $f'_c \ge 4.0$ ksi

$= 0.85$

$k = 0.28$

For Rectangular Section Behavior

$$c = \frac{64(0.153)(270)}{0.85(4)(0.85)(138) + (0.28)64(0.153)\dfrac{270}{(58.383)}} = 6.425 \text{ inches}$$

$a = 0.85 \times 6.425 = 5.461$ inches < 8 inches

Thus, its a rectangular section behavior.

$$f_{ps} = 270\left(1 - 0.28\frac{6.425}{(58.383)}\right) = 261.68 \text{ ksi}$$

The development length is calculated as,

$$l_d \ge 2.0\left(261.68 - \frac{2}{3}156.28\right)0.5 = 157.5 \text{ in.}$$

l_d = 13.12 ft.

Hence, the debonding length is the lesser of the following,

a. 15 ft.

b. 0.2 × 108.417 = 21.68 ft.

c. 0.5 × 108.417 - 13.12 = 41 ft.

Hence, the maximum debonding length to which the strands can be debonded is 15 ft.

Table B.2.6 Calculation of Initial Stresses at Extreme Fibers and Corresponding Required Initial Concrete Strengths

	Location of the Debonding Section (ft. from end)						
	End	3	6	9	12	15	Midspan
Row No. 1 (bottom row)	27	27	27	27	27	27	27
Row No. 2	27	27	27	27	27	27	27
Row No. 3	10	10	10	10	10	10	10
No. of Strands	64	64	64	64	64	64	64
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1,807.56	1,807.56	1,807.56	1,807.56	1,807.56	1,807.56	1,807.56
ec (in.)	18.743	18.743	18.743	18.743	18.743	18.743	18.743
Top Fiber Stresses (ksi)	-1.041	-0.867	-0.704	-0.550	-0.406	-0.272	0.571
Corresponding $f'_{ci\ reqd}$ (psi)	18,814	13,050	8,604	5,252	2,862	1,284	5,660
Bottom Fiber Stresses (ksi)	3.494	3.371	3.255	3.146	3.044	2.949	2.352
Corresponding $f'_{ci\ reqd}$ (psi)	5,823	5,618	5,425	5,243	5,074	4,915	3,920

In Table B.2.6, the calculation of initial stresses at the extreme fibers and corresponding requirement of f'_{ci} suggests that the preliminary estimate of f'_{ci} to be 4,000 psi is inadequate. Since strand can not be debonded beyond the section located at 15 ft. from the end of the beam, so, f'_{ci} is increased from 4,000 psi to 4,915 psi and at all other section, where debonding can be done, the strands are debonded to bring the required f'_{ci} below 4,915 psi. Table B.2.7 shows the debonding schedule based on the procedure described earlier.

Table B.2.7 Debonding of Strands at Each Section

	Location of the Debonding Section (ft. from end)						
	End	3	6	9	12	15	Midspan
Row No. 1 (bottom row)	7	9	17	23	25	27	27
Row No. 2	19	27	27	27	27	27	27
Row No. 3	10	10	10	10	10	10	10
No. of Strands	36	46	54	60	62	64	64
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1,016.76	1,299.19	1,525.13	1,694.591	1,751.08	1,807.56	1,807.56
ec (in.)	18.056	18.177	18.475	18.647	18.697	18.743	18.743
Top Fiber Stresses (ksi)	-0.531	-0.517	-0.509	-0.472	-0.367	-0.272	0.571
Corresponding $f'_{ci\ reqd}$ (psi)	4,895	4,640	4,498	3,868	2,338	1,284	5,660
Bottom Fiber Stresses (ksi)	1.926	2.347	2.686	2.919	2.930	2.949	2.352
Corresponding $f'_{ci\ reqd}$ (psi)	3,211	3,912	4,477	4,864	4,884	4,915	3,920

B.2.7.2 Iteration 2

Following the procedure in iteration 1 another iteration is required to calculate prestress losses based on the new value of f'_{ci} = 4,915 psi. The results of this second iteration are shown in Table B.2.8

Table B.2.8 Results of iteration No. 2

	Trial #1	Trial # 2	Trial # 3	Units
No. of Strands	64	64	64	
ec	18.743	18.743	18.743	in
Δf_{pSR}	8	8	8	ksi
Assumed Initial Prestress Loss	8.841	8.369	8.423	%
P_{si}	1,807.59	1,816.91	1,815.92	kips
M_g	1,714.65	1,714.65	1,714.65	k - ft.
f_{cgp}	2.233	2.249	2.247	ksi
f_{ci}	4,915	4,915	4,915	psi
E_{ci}	4,250	4,250	4,250	ksi
Δf_{pES}	14.973	15.081	15.067	ksi
f_{cdp}	1.191	1.191	1.191	ksi
Δf_{pCR}	18.459	18.651	18.627	ksi
Δf_{pR1}	1.975	1.975	1.975	ksi
Δf_{pR2}	2.616	2.591	2.594	ksi
Calculated Initial Prestress Loss	8.369	8.423	8.416	%
Total Prestress Loss	46.023	46.298	46.263	ksi

B.2.7.2.1 Total Losses at Transfer

Total Initial losses = $\Delta f_{ES} + \Delta f_{R1}$ = 15.067 + 1.975 = 17.042 ksi

f_{si} = effective initial prestress = 202.5 – 17.042 = 185.458 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 64(0.153)(185.458) = 1,816.005 kips

B.2.7.2.2 Total Losses at Service Loads

Δf_{SH} = 8 ksi

Δf_{ES} = 15.067 ksi

Δf_{R2} = 2.594 ksi

Δf_{R1} = 1.975 ksi

Δf_{CR} = 18.519 ksi

Total final losses = 8 + 15.067 + 2.594 + 1.975 + 18.627 = 46.263 ksi

or $\dfrac{46.263(100)}{0.75(270)}$ =22.85%

f_{se} = effective final prestress = 0.75(270) – 46.263 = 156.237 ksi

P_{se} = 64 (0.153) (156.237) = 1,529.873 kips

B.2.7.2.3 Final Stresses at Midspan

Top fiber stress in concrete at midspan at service loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1529.873}{1120} - \frac{18.743(1529.873)}{12761.88} + 3.737$$

$= 1.366 - 2.247 + 3.737 = 2.856$ ksi

Allowable compression stress limit for all load combinations = 0.6 f'_c

$f'_{c\ reqd.} = 2856/0.6 = 4{,}760$ psi

Top fiber stress in concrete at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}$$

$$= \frac{1529.873}{1120} - \frac{18.743(1529.873)}{12761.88} + \frac{(1714.65 + 1689.67 + 132.63)(12)}{12761.88} + \frac{(160.15+283.57)(12)}{79936.06}$$

$= 1.366 - 2.247 + 3.326 + 0.067 = 2.512$ ksi

Allowable compression stress limit for effective pretension force + permanent dead loads = 0.45 f'_c

$f'_{c\ reqd.} = 2512/0.45 = 5{,}582$ psi (controls)

Top fiber stress in concrete at midspan due to live load + ½(effective prestress + dead loads)

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}\right)$$

$$= \frac{(1618.3+684.57)(12)}{79936.06} + 0.5\left(\begin{array}{l}\dfrac{1529.873}{1120} - \dfrac{18.743(1529.873)}{12761.88} \\ + \dfrac{(1714.65 + 1689.67+132.63)(12)}{12761.88} + \dfrac{(160.15+283.57)(12)}{79936.06}\end{array}\right)$$

$= 0.346 + 0.5(1.366 - 2.247 + 3.326 + 0.067) = 1.602$ ksi

Allowable compression stress limit for effective pretension force + permanent dead loads = 0.4 f_c'

$f_c'{}_{reqd.}$ = 1602/0.4 = 4,005 psi

Bottom fiber stress in concrete at midspan at service load

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - f_b$$

$$f_{bf} = \frac{1529.873}{1120} + \frac{18.743(1529.873)}{18024.15} - 3.34 = 1.366 + 1.591 - 3.34 = -0.383 \text{ ksi}$$

Allowable tension in concrete = $0.19\sqrt{f_c'(ksi)}$

$$f_c'{}_{reqd.} = \left(\frac{383}{0.19}\right)^2 \times 1000 = 4{,}063 \text{ psi}$$

B.2.7.2.4 Initial Stresses at Debonding Locations

With the same number of debonded strands, as was determined in the previous iteration, the top and bottom fiber stresses with their corresponding initial concrete strengths are calculated. It can be observed that at 15 ft. location, the f_{ci}' value is updated to 4943 psi.

Table B.2.9 Debonding of Strands at Each Section

	Location of the Debonding Section (ft. from end)						
	0	3	6	9	12	15	54.2
Row No. 1 (bottom row)	7	9	17	23	25	27	27
Row No. 2	19	27	27	27	27	27	27
Row No. 3	10	10	10	10	10	10	10
No. of Strands	36	46	54	60	62	64	64
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1,021.50	1,305.25	1,532.25	1,702.50	1,759.26	1,816.01	1,816.01
ec (in.)	18.056	18.177	18.475	18.647	18.697	18.743	18.743
Top Fiber Stresses (ksi)	-0.533	-0.520	-0.513	-0.477	-0.372	-0.277	0.567
Corresponding $f_{ci\ reqd}'$ (psi)	4,932	4,694	4,569	3,950	2,403	1,332	5,581
Bottom Fiber Stresses (ksi)	1.935	2.359	2.700	2.934	2.946	2.966	2.368
Corresponding $f_{ci\ reqd}'$ (psi)	3,226	3,931	4,500	4,890	4,910	4,943	3,947

B.2.7.3 Iteration 3

Following the procedure in iteration 1, a third iteration is required to calculate prestress losses based on the new value of f'_{ci} = 4943 psi. The results of this second iteration are shown in Table B.2.10

Table B.2.10 Results of iteration No. 3

	Trial #1	Trial #2	Units
No. of Strands	64	64	
ec	18.743	18.743	
Δf_{pSR}	8	8	in.
Assumed Initial Prestress Loss	8.416	8.395	ksi
P_{si}	1,815.922	1,816.516	%
M_g	1,714.65	1,714.65	kips
f_{cgp}	2.247	2.248	k - ft.
f_{ci}	4943.000	4943.000	ksi
E_{ci}	4262.321	4262.321	psi
Δf_{pES}	15.025	15.031	ksi
f_{cdp}	1.191	1.191	ksi
Δf_{pCR}	18.627	18.639	ksi
Δf_{pR1}	1.975	1.975	ksi
Δf_{pR2}	2.599	2.598	ksi
Corresponding Initial Prestress Loss	8.395	8.398	ksi
Total Prestress Loss	46.226	46.243	%

B.2.7.3.1 Total Losses at Transfer

Total Initial losses = $\Delta f_{ES} + \Delta f_{R1}$ = 15.031 + 1.975 = 17.006 ksi

f_{si} = effective initial prestress = 202.5 – 17.006= 185.494 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 64(0.153)(185.494) = 1,816.357 kips

B.2.7.3.2 Total Losses at Service Loads

Δf_{SH} = 8 ksi

Δf_{ES} = 15.031 ksi

Δf_{R2} = 2.598 ksi

Δf_{R1} = 1.975 ksi

Δf_{CR} = 18.639 ksi

Total final losses = 8 + 15.031 + 2.598 + 1.975 + 18.639 = 46.243 ksi

$$\text{or } \frac{46.243\ (100)}{0.75(270)} = 22.84\%$$

f_{se} = effective final prestress = 0.75(270) – 46.243 = 156.257 ksi

P_{se} = 64(0.153)(156.257) = 1,530.069 kips

B.2.7.3.3 Final Stresses at Midspan

Top fiber stress in concrete at midspan at service loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1530.069}{1120} - \frac{18.743(1530.069)}{12761.88} + 3.737$$

$$= 1.366 - 2.247 + 3.737 = 2.856 \text{ ksi}$$

Allowable compression stress limit for all load combinations = 0.6 f'_c

$f'_{c\ reqd.}$ = 2856/0.6 = 4,760 psi

Top fiber stress in concrete at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}$$

$$= \frac{1530.069}{1120} - \frac{18.743(1530.069)}{12761.88} + \frac{(1714.65 + 1689.67 + 132.63)(12)}{12761.88} + \frac{(160.15 + 283.57)(12)}{79936.06}$$

$$= 1.366 - 2.247 + 3.326 + 0.067 = 2.512 \text{ ksi}$$

Allowable compression stress limit for effective pretension force + permanent dead loads = 0.45 f'_c

$f'_{c\ reqd.}$ = 2512/0.45 = 5,582 psi (controls)

Top fiber stress in concrete at midspan due to live load + ½(effective prestress + dead loads)

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}\right)$$

$$= \frac{(1618.3 + 684.57)(12)}{79936.06} + 0.5\left(\begin{array}{l}\frac{1530.069}{1120} - \frac{18.743(1530.069)}{12761.88} + \\ \frac{(1714.65 + 1689.67 + 132.63)(12)}{12761.88} + \frac{(160.15 + 283.57)(12)}{79936.06}\end{array}\right)$$

$$= 0.346 + 0.5(1.366 - 2.247 + 3.326 + 0.067) = 1.602 \text{ ksi}$$

Allowable compression stress limit for effective pretension force + permanent dead loads = 0.4 f'_c

$f'_{c\ reqd.}$ = 1602/0.4 = 4,005 psi

Bottom fiber stress in concrete at midspan at service load

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\, e_c}{S_b} - f_b$$

$$f_{bf} = \frac{1530.069}{1120} + \frac{18.743(1530.069)}{18024.15} - 3.34 = 1.366 + 1.591 - 3.34 = -0.383 \text{ ksi}$$

Allowable tension in concrete = $0.19\sqrt{f'_c(ksi)}$

$$f'_{c\ reqd.} = \left(\frac{383}{0.19}\right)^2 \times 1000 = 4{,}063 \text{ psi}$$

B.2.7.3.4 Initial Stresses at Debonding Location

With the same number of debonded strands, as was determined in the previous iteration, the top and bottom fiber stresses with their corresponding initial concrete strengths are calculated. It can be observed that at 15 ft. location, the f'_{ci} value is updated to 4944 psi.

Table B.2.11 Debonding of Strands at Each Section

	Location of the Debonding Section (ft. from end)						
	0	3	6	9	12	15	54.2
Row No. 1 (bottom row)	7	9	17	23	25	27	27
Row No. 2	19	27	27	27	27	27	27
Row No. 3	10	10	10	10	10	10	10
No. of Strands	36	46	54	60	62	64	64
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1021.70	1305.51	1532.55	1702.84	1759.60	1816.36	1816.36
ec (in.)	18.056	18.177	18.475	18.647	18.697	18.743	18.743
Top Fiber Stresses (ksi)	-0.533	-0.520	-0.513	-0.477	-0.372	-0.277	0.566
Corresponding $f'_{ci\ reqd}$ (psi)	4932	4694	4569	3950	2403	1332	5562
Bottom Fiber Stresses (ksi)	1.936	2.359	2.701	2.934	2.947	2.966	2.369
Corresponding $f'_{ci\ reqd}$ (psi)	3226	3932	4501	4891	4911	4944	3948

Since in the last iteration, actual initial losses are 8.398% as compared to previously assumed 8.395% and f'_{ci} = 4,944 psi as compared to previously assumed f'_{ci} = 4,943 psi. These values are close enough, so no further iteration will be required. Use f'_c = 5,582 psi, f'_{ci} = 4,944 psi

B.2.8
STRESS SUMMARY
B.2.8.1
Concrete Stresses at Transfer
B.2.8.1.1
Allowable Stress Limits

Compression: 0.6 f'_{ci} = 0.6(4944) = +2,966.4 psi = 2.966 ksi (compression)

Tension:

The maximum allowable tensile stress for bonded reinforcement (precompressed tensile zone) is

$0.24\sqrt{f'_{ci}}$ = $[0.24\sqrt{4.944(\text{ksi})}\ ]\times 1000 = 534$ psi

The maximum allowable tensile stress for without bonded reinforcement (non-precompressed tensile zone) is

$0.0948\sqrt{f'_{ci}}$ =$[0.0948\times\sqrt{4.944(\text{ksi})}\ ]\times 1000 = 210.789$ ksi ≥ 0.2 ksi

B.2.8.1.2
Stresses at Beam End and at Transfer Length Section

Stresses at beam end and transfer length section need only be checked at release, because losses with time will reduce the concrete stresses making them less critical.

B.2.8.1.2.1
Stresses at Transfer Length Section

Transfer length = 60 (strand diameter) [LRFD Art. 5.8.2.3]
= 60 (0.5) = 30 in. = 2.5 ft.

Transfer length section is located at a distance of 2.5 ft. from end of the beam. Overall beam length of 109.5 ft. is considered for the calculation of bending moment at transfer length. As shown in Table B.2.11, the number of strands at this location, after debonding of strands, is 36.

Moment due to beam self-weight and diaphragm,

M_g = 0.5(1.167) (2.5) (109.5 – 2.5) = 156.086 k –ft.

M_{dia} = 3(2.5) = 7.5 k–ft.

Concrete stress at top fiber of the beam

$$f_t = \frac{P_{si}}{A} - \frac{P_{si}\,e_t}{S_t} + \frac{M_g + M_{dia}}{S_t}$$

P_{si} = 36 (0.153) (185.494) = 1,021.701 kips

Strand eccentricity at transfer section, e_c = 18.056 in.

$$f_t = \frac{1021.701}{1120} - \frac{18.056(1021.701)}{12761.88} + \frac{(156.086+7.5)(12)}{12761.88}$$

= 0.912 – 1.445 + 0.154 = -0.379 ksi

Allowable tension (with bonded reinforcement) = 534 psi > 379 psi (O.K.)

Concrete stress at the bottom fiber of the beam

$$f_b = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g + M_{dia}}{S_b}$$

$$f_{bi} = \frac{1021.701}{1120} + \frac{18.056(1021.701)}{18024.15} - \frac{(156.086+7.5)(12)}{18024.15}$$

$= 0.912 + 1.024 – 0.109 = 1.827$ ksi

Allowable compression = 2.966 ksi > 1.827 ksi (reqd.) (O.K.)

B.2.8.1.2.2
Stresses at Beam End

And the strand eccentricity at end of beam is:

$$e_c = 22.36 - \frac{7(2.17)+17(4.14)+8(6.11)}{36} = 18.056 \text{ in.}$$

$P_{si} = 36\ (0.153)\ (185.494) = 1{,}021.701$ kips

Concrete stress at the top fiber of the beam

$$f_t = \frac{1021.701}{1120} - \frac{18.056(1021.701)}{12761.88} = 0.912 - 1.445 = -0.533 \text{ ksi}$$

Allowable tension (with bonded reinforcement) = -0.534 psi > -0.533 psi (O.K.)

Concrete stress at the bottom fiber of the beam

$$f_b = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1021.701}{1120} + \frac{18.056(1021.701)}{18024.15} = 0.912 + 1.024 = 1.936 \text{ ksi}$$

Allowable compression = 2.966 ksi > 1.936 ksi (reqd.) (O.K.)

B.2.8.1.3
Stresses at Midspan

Bending moment at midspan due to beam self –weight based on overall length

$M_g = 0.5(1.167)(54.21)(109.5 – 54.21) = 1{,}749.078$ K-ft.

$P_{si} = 64\ (0.153)\ (185.494) = 1{,}816.357$ kips

Concrete stress at top fiber of the beam at midspan

$$f_t = \frac{P_{si}}{A} - \frac{P_{si}\,e_c}{S_t} + \frac{M_g}{S_t}$$

$$f_t = \frac{1816.357}{1120} - \frac{18.743(1816.357)}{12761.88} + \frac{1749.078(12)}{12761.88}$$

= 1.622 – 2.668 + 1.769 = 0.723 ksi

Allowable compression: 2.966 ksi >> 0.723 ksi (reqd.) (O.K.)

Concrete stresses in bottom fibers of the beam at midspan

$$f_b = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_b = \frac{1816.357}{1120} + \frac{18.743(1816.357)}{18024.15} - \frac{1749.078\,(12)}{18024.15}$$

= 1.622 + 1.889 - 1.253 = 2.258 ksi

Allowable compression: 2.966 ksi > 2.258 ksi (reqd.) (O.K.)

B.2.8.1.4 Stress Summary at Transfer

	Top of beam f_t (ksi)	Bottom of beam f_b (ksi)
At End	-0.533	+1.936
At transfer length section	-0.379	+1.827
At Midspan	+0.723	+2.258

B.2.8.2 Concrete Stresses at Service Loads

B.2.8.2.1 Allowable Stress Limits

Compression

Case (I): for all load combinations

$0.60\,f_c'$ = 0.60(5582)/1000 = +3.349 ksi (for precast beam)

$0.60\,f_c'$ = 0.60(4000)/1000 = +2.4 ksi (for slab)

Case (II): for effective pretension force + permanent dead loads

$0.45\,f_c'$ = 0.45(5582)/1000 = +2.512 ksi (for precast beam)

$0.45\,f_c'$ = 0.45(4000)/1000 = +1.8 ksi (for slab)

Case (III): for live load +1/2(effective pretension force + dead loads)

$0.40\,f_c'$ = 0.40(5582)/1000 = +2.233 ksi (for precast beam)

$0.40\,f_c'$ = 0.40(4000)/1000 = +1.6 ksi (for slab)

Tension: $0.19\sqrt{f_c'} = 0.19\sqrt{5.582(\text{ksi})}\times 1000 = -448.9$ ksi

$P_{se} = 64(0.153)(156.257) = 1{,}530.069$ kips

B.2.8.2.2 Stresses at Midspan

Case (I): Concrete stresses at top fiber of the beam at service loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1530.069}{1120} - \frac{18.743(1530.069)}{12761.88} + 3.737$$

$$= 1.366 - 2.247 + 3.737 = 2.856 \text{ ksi}$$

Allowable compression: +3.349 ksi > +2.856 ksi (reqd.) (O.K.)

Case (II): Effective pretension force + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}$$

$$= \frac{1530.069}{1120} - \frac{18.743(1530.069)}{12761.88} + \frac{(1714.65 + 1689.67 + 132.63)(12)}{12761.88} + \frac{(160.15 + 283.57)(12)}{79936.06}$$

$$= 1.366 - 2.247 + 2.326 + 0.067 = 1.512 \text{ ksi}$$

Allowable compression: +2.512 ksi > +1.512 ksi (reqd.) (O.K.)

Case (III): Live load + ½(Pretensioning force + dead loads)

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_b + M_{dia}}{S_t} + \frac{M_b + M_{ws}}{S_{tg}}\right)$$

$$= \frac{(1618.3 + 684.57)(12)}{79936.06} + 0.5\left(\frac{1525.956}{1120} - \frac{18.743(1525.956)}{12761.88} + \frac{(1714.65 + 1689.67 + 132.63)(12)}{12761.88} + \frac{(160.15 + 283.57)(12)}{79936.06}\right)$$

$$= 0.346 + 0.5(1.366 - 2.247 + 2.326 + 0.067) = 1.602 \text{ ksi}$$

Allowable compression: +2.233 ksi > +1.602 ksi (reqd.) (O.K.)

Concrete stresses at bottom fiber of the beam:

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - f_b$$

$$f_{bf} = \frac{1530.069}{1120} + \frac{18.743(1530.069)}{18024.15} - 3.34 = 1.366 + 1.591 - 3.338 = -0.383 \text{ ksi}$$

Allowable Tension: -0.449 ksi (O.K.)

B.2.8.2.3
Stresses at the Top of the Deck Slab

Stresses at the top of the slab

Case (I):

$$f_t = \frac{M_b + M_{ws} + M_{LT} M_{LL}}{S_{tc}} = \frac{(1618.3+684.57+160.15+283.57)(12)}{50802.19}$$

$$= +0.649 \text{ ksi}$$

Allowable compression: +2.4 ksi > +0.649 ksi (reqd.) (O.K.)

Case (II):

$$f_t = \frac{M_b + M_{ws}}{S_{tc}} = \frac{(160.15+283.57)(12)}{50802.19} = 0.105 \text{ ksi}$$

Allowable compression: +1.8 ksi > +0.105 ksi (reqd.) (O.K.)

Case (III):

$$f_t = \frac{0.5(M_b + M_{ws}) + M_{LT} M_{LL}}{S_{tc}} = \frac{(1618.3+684.57+0.5(160.15+283.57))(12)}{50802.19}$$

$$= 0.596 \text{ ksi}$$

Allowable compression: +1.6 ksi > +0.596 ksi (reqd.) (O.K.)

B.2.8.2.4
Summary of Stresses at Service Loads

		Top of Slab f_t (ksi)	Top of Beam f_t (ksi)	Bottom of Beam f_b (ksi)
At Midspan	CASE I	+ 0.649	+2.856	
	CASE II	+ 0.105	+1.512	-0.383
	CASE III	+0.596	+1.602	

B.2.8.3
Fatigue Stress Limit

According to LRFD Art. 5.5.3, the fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit. Since, in this detailed design example the U54 girder is being designed as a fully prestressed component and the extreme fiber tensile stress due to Service III Limit State is within the allowable tensile stress limits, no fatigue check is required.

B.2.8.4 Actual Modular Ratio and Transformed Section Properties for Strength Limit State and Deflection Calculations

Till this point, a modular ratio equal to 1 has been used for the Service Limit State design. For the evaluation of Strength Limit State and Deflection calculations, actual modular ratio will be calculated and the transformed section properties will be used.

$$n = \left(\frac{E_c \text{ for slab}}{E_c \text{ for beam}}\right) = \left(\frac{3834.25}{4341.78}\right) = 0.846$$

Transformed flange width = n (effective flange width) = 0.846(138 in.)
= 116.75 in.

Transformed Flange Area = n (effective flange width) (t_s) = 1(116.75 in.)(8 in.)
= 934 in.^{2}

Table B.2.12 Properties of Composite Section

	Transformed Area in.^{2}	y_b in.	$A\,y_b$ in.	$A(y_{bc} - y_b)^2$	I in.^{4}	$I+A(y_{bc}- y_b)^2$ in.^{4}
Beam	1,120	22.36	25,043.20	294,295.79	403,020	697,315.79
Slab	934	58	54,172.00	352,608.26	4,981	357,589.59
$\sum$	2,054		79,215.20			1,054,905.38

A_c = total area of composite section = 2,054 in.^{2}

h_c = total height of composite section = 62 in.

I_c = moment of inertia of composite section = 1,054,905.38 in.^{4}

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam = 79,215.20 / 2,054 = 38.57 in.

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam = 54 – 38.57 = 15.43 in.

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab = 62 – 38.57 = 23.43 in.

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam
= I_c/y_{bc} = 1,054,905.38 / 38.57 = 27,350.41 in.^{3}

S_{tg} = composite section modulus for top fiber of the precast beam
= I_c/y_{tg} = 1,054,905.38 / 15.43 = 68,367.17 in.^{3}

S_{tc} = composite section modulus for top fiber of the slab
= I_c/y_{tc} = 1,054,905.38 / 23.43 = 45,023.7 in.^{3}

B.2.9 STRENGTH LIMIT STATE

Total ultimate moment from strength I is:

$$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM)$$

$$M_u = 1.25(1714.65 + 1689.67 + 132.63 + 160.15) + 1.5(283.57)$$
$$+ 1.75(1618.3 + 684.57) = 9{,}076.73 \text{ k - ft}$$

Average stress in prestressing steel when $f_{pe} \geq 0.5\, f_{pu}$ = (156.257 > 0.5(270) = 135 ksi)

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ [LRFD Eq. 5.7.3.1.1-1]

k = 0.28 for low-relaxation strand [LRFD Table C5.7.3.1.1-1]

For Rectangular Section Behavior

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s' f_y'}{0.85 f_c' \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1-4]

$d_p = h - y_{bs} = 62 - 3.617 = 58.383$ in.

$\beta_1 = 0.85$ for $f'_c \leq 4.0$ ksi [LRFD Art. 5.7.2.2]

$= 0.85 - 0.05(f'_c - 4.0) \leq 0.65$ for $f'_c \geq 4.0$ ksi

$= 0.85$

$k = 0.28$

For rectangular section behavior

$$c = \frac{64(0.153)(270)}{0.85(5.587)(0.85)(116.75) + (0.28)64(0.153)\dfrac{270}{(58.383)}} = 5.463 \text{ inches}$$

$a = 0.85 \times 5.463 = 4.64$ inches < 8 inches

Thus, its a rectangular section behavior.

$$f_{ps} = 270\left(1 - 0.28\frac{5.463}{(58.383)}\right) = 262.93 \text{ ksi}$$

Nominal flexural resistance, [LRFD Art. 5.7.3.2.3]

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$ [LRFD Eq. 5.7.3.2.2-1]

The equation above is a simplified form of LRFD Equation 5.7.3.2.2-1 because no compression reinforcement or mild tension reinforcement is considered and the section behaves as a rectangular section.

$$M_n = 64(0.153)(262.93)\left(58.383 - \frac{4.64}{2}\right)$$

$$= 144{,}340.39 \text{ k - in} = 12{,}028.37 \text{ k - ft}$$

Factored flexural resistance:

$M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where,

ϕ = resistance factor [LRFD Eq. 5.5.4.2.1]

$= 1.00$, for flexure and tension of prestress concrete

$M_r = 12028.37$ k – ft. $> M_u = 9076.73$ k – ft. (O.K.)

B.2.9.1 LIMITS OF REINFORCEMENT

B.2.9.1.1 Maximum Reinforcement

[LRFD Eq. 5.7.3.3]

The amount of prestressed and non-prestressed reinforcement should be such that

$$\frac{c}{d_e} \le 0.42$$ [LRFD Eq. 5.7.3.3.1-1]

where, $$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$$ [LRFD Eq. 5.7.3.3.1-2]

Since $A_s = 0$, $d_e = d_p = 58.383$ in.

$$\frac{c}{d_e} = \frac{5.463}{58.383} = 0.094 \le 0.42$$ O.K.

[LRFD Art. 5.7.3.3.2]

B.2.9.1.2 Minimum Reinforcement

At any section, the amount of prestressed and nonprestressed tensile reinforcement should be adequate to develop a factored flexural resistant, M_r, equal to the lesser of:

- 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, and,
- 1.33 times the factored moment required by the applicable strength load combination.

Check at the midspan:

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \le S_c f_r$$ [LRFD Eq. 5.7.3.3.2-1]

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

$$f_{cpe} = \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b} = \frac{1530.069}{1120} + \frac{1530.069(18.743)}{18024.15} = 1.366 + 1.591 = 2.957\ ksi$$

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-ft.)

$= M_g + M_{slab} + M_{dia}$ = 1714.65+1689.67+132.63 = 3536.95 kip-ft.

$S_c = S_{bc}$

$S_{nc} = S_b$

f_r = $f_r = 0.24\sqrt{f'_c(ksi)} = 0.24(\sqrt{5.587}) = 0.567\ ksi$ [LRFD Art. 5.4.6.2]

$$M_{cr} = \frac{27350.41}{12}(0.567 + 2.957) - 3536.95\left(\frac{27350.41}{18024.15} - 1\right) \le \frac{27350.41}{12}(0.567)$$

$M_{cr} = 6183.54 \le 1292.31$

so use $M_{cr} = 1,292.31$ k-ft

$1.2M_{cr} = 1,550.772$ k-ft

where, $M_u = 9,076.73$ k-ft

$1.33M_u = 12,097.684$ k-ft

Since $1.2M_{cr} < 1.33\ M_u$, the $1.2\ M_{cr}$ requirement controls.

$M_r = 12{,}028.37$ k – ft. $> 1.2M_{cr} = 1{,}550.772$ k-ft. O.K.

Art. 5.7.3.3.2 LRFD Specifications require that this criterion be met at every section.

B.2.10 TRANSVERSE SHEAR DESIGN

The area and spacing of shear reinforcement must be determined at regular intervals along the entire length of the beam. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports.

Transverse shear reinforcement is provided when:

$V_u < 0.5\ \phi\,(V_c + V_p)$ [LRFD Art. 5.8.2.4-1]

where,

V_u = the factored shear force at the section considered

V_c = the nominal shear strength provided by concrete

V_s = the nominal shear strength provided by web reinforcement

ϕ = strength reduction factor = 0.90 [LRFD Art. 5.5.4.2.1]

B.2.10.1 Critical Section

Critical section near the supports is the greater of: [LRFD Art. 5.8.3.2]

$0.5d_v\cot\theta$ or d_v

Where

d_v = effective shear depth

= distance between resultants of tensile and compressive forces, (d_e - $a/2$), but not less than the greater of ($0.9d_e$) or ($0.72h$) [LRFD Art. 5.8.2.9]

θ = angle of inclination of diagonal compressive stresses, assume θ is 23° (slope of compression field)

B.2.10.1.1 Angle of Diagonal Compressive Stresses

The shear design at any section depends on the angle of diagonal compressive stresses at the section. Shear design is an iterative process that begins with assuming a value for θ.

B.2.10.1.2 Effective Shear Design

d_v = $d_e - a/2 = 58.383 - 4.64/2 = 56.063$ in. (controls)

$0.9\ d_e = 0.9\ (58.\ 383) = 52.545$ in.

$0.72h = 0.72 \times 62 = 44.64$ in.

B.2.10.1.3 Calculation of Critical Section

The critical section near the support is greater of:
d_v = 56.063 in.

and

$0.5d_v\cot\theta$ = 0.5×(56.063)×cot(23) = 66.04 in. = 5.503 ft. (controls)

B.2.10.2 Contribution of Concrete to Nominal Shear Resistance

The contribution of the concrete to the nominal shear resistance is:

$$V_c = 0.0316\beta\sqrt{f_c'(ksi)}b_v d_v$$ [LRFD Eq. 5.8.3.3-3]

B.2.10.2.1 Strain in Flexural Tension Reinforcement

Calculate the strain in the reinforcement on the flexural tension side. Assuming that the section contains at least the minimum transverse reinforcement as specified in LRFD Specifications Article 5.8.2.5:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \leq 0.001$$ [LRFD Eq. 5.8.3.3-1]

If LRFD Eq. 5.8.3.3-1 yield a negative value, then, LRFD Eq. 5.8.3.3-3 should be used given as below:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})}$$ [LRFD Eq. 5.8.3.3-3]

where,

V_u = factored shear force at the critical section, taken as positive quantity

= 1.25(56.84+56.01+3.00+5.31)+1.50(9.40)+1.75(85.55+32.36) = 371.893 kips

M_u = 1.25(330.46+325.64+16.51+30.87)+1.5(54.65)+1.75(331.15+131.93)

M_u = factored moment, taken as positive quantity 1771.715 k-ft. > V_ud_v (kip-in.)

= 1771.715 k – ft. > 371.893×56.063/12 = 1,737.45 kip – ft. O.K.

V_p = component of the effective prestressing force in the direction of the applied shear = 0 (because no harped strands are used)

N_u = applied factored normal force at the specified section = 0

A_c = area of the concrete (in.^{2}) on the flexural tension side below $h/2$ = 714 in.^{2}

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{371.893}{0.9 \times 10 \times 56.063} = 0.737 \text{ ksi} \qquad \text{[LRFD Eq. 5.8.2.9-1]}$$

v_u / f'_c = 0.737 / 5.587 = 0.132

As per LRFD Art. 5.8.3.4.2, if the section is within the transfer length of any strands, then calculate the effective value of f_{po}, else assume $f_{po} = 0.7 f_{pu}$

Since, transfer length of the bonded strands at the section located at 3 ft. from the end of the beam extends from 3 ft. to 5.5 ft. from the end of the beam, whereas the critical section for shear is 5.47 ft. from the support center line. The support center line is 6.5 in. away from the end of the beam. The critical section for shear will be 5.47 + 6.5/12 = 6.00 ft. from the end of the beam, so the critical section does not fall within the transfer length of the strands that are bonded from the section located at 3 ft. from the end of the beam, thus, we do not need to perform detailed calculations for f_{po}.

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplifed by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi).
= approximately equal to 0.7 f_{pu} [LRFD Fig. C5.8.3.4.2-5]
= 0.70 f_{pu} = 0.70×270 = 189 ksi

Or it can be conservatively taken as the effective stress in the prestressing steel, f_{pe}

$$f_{po} = f_{pe} + f_{pc}\left(\frac{E_{ps}}{E_c}\right)$$

where,

f_{pc} = Compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange (ksi); in a composite section, it is the resultant compressive stress at the centroid of the composite section or at the junction of the web and flange when the centroid lies within the flange, that results from both prestress and the bending moments resisted by the precast member acting alone (ksi).

$$f_{pc} = \frac{P_{se}}{A_n} - \frac{P_{se} ec\left(y_{bc} - y_b\right)}{I} + \frac{\left(M_g + M_{slab}\right)\left(y_{bc} - y_b\right)}{I}$$

The number of strands at the critical section location is 46 and the corresponding eccentricity is 18.177 in., as calculated in Table B.2.11.

$$P_{se} = 46 \times 0.153 \times 155.837 = 1,096.781 \text{ ksi}$$

$$f_{pc} = \frac{1096.781}{1120} - \frac{1096.781 \times 18.177(40.05 - 22.36)}{403020} + \frac{12 \times (328.58 + 323.79)(40.05 - 22.36)}{403020} = 0.492 \text{ ksi}$$

$$f_{po} = 155.837 + 0.492\left(\frac{28500}{4531.48}\right) = 158.93 \text{ ksi}$$

$$\varepsilon_x = \frac{\frac{1771.715 \times 12}{56.063} + 0.5(0.0) + 0.5(371.893 - 0.0)\cot 23^o - 46 \times 0.153 \times 158.93}{2(28000 \times 0.0 + 28500 \times 46 \times 0.153)} \leq 0.001$$

B.2.10.2.2
Values of β and θ

ε_x = -7.51 x 10^{-04} ≤ 0.001

Since this value is negative LRFD Eq. 5.8.3.4.2-3 should be used to calculate ε_x

$$\varepsilon_x = \frac{\frac{1771.715 \times 12}{56.063} + 0.5(0.0) + 0.5(371.893 - 0.0)\cot 23^o - 46 \times 0.153 \times 158.93}{2((4531.48)(714) + ((28500)(46)(0.153))}$$

ε_x = -4.384 x 10^{-05}

b_v = 2×5 in. = 10 in. [LRFD Art. 5.8.2.9]

Choose the values of β and θ from LRFD Table 5.8.3.4.2-1 and after interpolation We get the final values of β and θ, as shown in Table B.2.13. Since θ = 23.3° value is close to the 23° assumed, no further iterations are required.

Table B.2.13 Interpolation for β and θ

v_u / f'_c	ε_x x 1000		
	-0.05	**-0.04384**	0
0.15	24.2		25
	2.776		2.72
0.132	23.19	**θ = 23.3**	24.06
	2.895	**β = 2.89**	2.83
0.125	22.8		23.7
	2.941		2.87

B.2.10.2.3 Concrete Contribution

The nominal shear resisted by the concrete is:

$$V_c = 0.0316\beta\sqrt{f_c'(ksi)}b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

$$V_c = 0.0316(2.89)\sqrt{5.587}(56.063)(10) = 121.02 \text{ kips}$$

B.2.10.3 Contribution of Reinforcement to Nominal Shear Resistance

B.2.10.3.1 Requirement for Reinforcement

Check if $V_u > 0.5\ \phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

V_u = 371.893 > 0.5×0.9×(121.02+0) = 54.46 kips

Therefore, transverse shear reinforcement should be provided.

B.2.10.3.2 Required Area of Reinforcement

$$\frac{V_u}{\phi} \le V_n = (V_c + V_s + V_p) \qquad \text{[LRFD Eq. 5.8.3.3-1]}$$

V_s = shear force carried by transverse reinforcement

$$= \frac{V_u}{\phi} - V_c - V_p = \left(\frac{371.893}{0.9} - 121.02 - 0\right) = 292.19 \text{ kips}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad \text{[LRFD Eq. 5.8.3.3-4]}$$

where s = spacing of stirrups, in.

α = angle of inclination of transverse reinforcement to longitudinal axis = 90^0

Therefore, area of shear reinforcement within a spacing s is:

reqd $A_v = (s\ V_s)/(f_y d_v \cot\theta)$

$= (s \times 292.19)/(60 \times 56.063 \times \cot(23)) = 0.0369 \times s$

If s = 12 in., then A_v = 0.443 in.^{2} / ft.

B.2.10.3.2 Spacing of Reinforcement

Maximum spacing of transverse reinforcement may not exceed the following:

Since v_u =0.737 > 0.125× f_c' = 0.125×5.587 = 0.689 [LRFD Art.. 5.8.2.7]

So, s_{max} = 0.4× 56.063 = 22.43 in. < 24.0 in. use s_{max} = 22.43 in.

Use 1 # 4 double legged with A_v = 0.392 in.[2] / ft., the required spacing can be calculated as,

$$s = \frac{A_v}{0.0369} = \frac{0.392}{0.0369} = 10.6 \text{ in.}$$

$$V_s = \frac{0.392(60)(56.063)(\cot 23)}{10}$$

$$= 310.643 \text{ kips} > V_s(\text{reqd.}) = 292.19 \text{ kips}$$

B.2.10.3.3 Minimum Reinforcement Requirement

[LRFD Art.. 5.8.2.5]

The area of transverse reinforcement should be less than:

$$A_s \geq 0.0316\sqrt{f_c'(ksi)}\frac{b_v s}{f_y} \quad \text{[LRFD Eq. 5.8.2.5-1]}$$

$$A_s \geq 0.0316\sqrt{5.587}\frac{10\times10}{60} = 0.125 \text{ in.}^2 \qquad \text{O.K.}$$

B.2.10.3.4 Maximum Nominal Shear Reinforcement

In order to assure that the concrete in the web of the beam will not crush prior to yield of the transverse reinforcement, the LRFD Specifications give an upper limit for V_n as follows:

$$V_n = 0.25 f_c' b_v d_v + V_p \quad \text{[LRFD Eq. 5.8.3.3-2]}$$

$$V_c + V_s \leq 0.25 f_c' b_v d_v + V_p$$

$$(121.02 + 310.643) < (0.25\times5.587\times10\times56.063+0)$$

$$431.663 \text{ kips} < 783.06 \text{ kips} \quad \text{O.K.}$$

B.2.10.4 Minimum Longitudinal Reinforcement Requirement

Longitudinal reinforcement should be proportioned so that at each section the following equation is satisfied:

$$A_s f_y + A_{ps} f_{ps} \geq \frac{M_u}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\frac{V_u}{\phi_v} + 0.5V_s - V_p\right)\cot\theta \quad \text{[LRFD Eq. 5.8.3.5-1]}$$

Using load combination Strength I, the factored shear force and bending moment at the face of bearing:

V_u = 1.25(62.82+61.91+3+5.87)+1.5(10.39)+1.75(90.24+35.66) = 402.91 kips

M_u = 1.25(23.64+23.3+1.13+2.2)+1.5(3.91)+1.75(23.81+9.44) = 126.885 k -ft.

$$46\times0.153\times262.93 \geq \frac{126.885\times12}{56.063\times1.0} + 0.0 + \left(\frac{402.91}{0.9} + 0.5\times310.643 - 0.0\right)\cot 23$$

1850.5 $\geq$ 1448.074 (O.K.)

B.2.11 INTERFACE SHEAR TRANSFER

[LRFD Art. 5.8.4]

B.2.11.1 Factored Horizontal Shear

According to the guidance given by the LRFD Specifications for computing the factored horizontal shear.

$$V_h = \frac{V_u}{d_e}$$ [LRFD Eq. C5.8.4.1-1]

V_h = horizontal shear per unit length of girder, kips

V_u = the factored vertical shear, kips

d_e = the distance between the centroid of the steel in the tension side of the beam to the center of the compression blocks in the deck (d_e - $a/2$), in

The LRFD Specifications do not identify the location of the critical section. For convenience, it will be assumed here to be the same location as the critical section for vertical shear, i.e. 5.503 ft. from the support center line.

V_u = 1.25(5.31)+1.50(9.40)+1.75(85.55+32.36) = 227.08 kips
d_e = 58.383 – 4.64/2 = 56.063 in.

$$V_h = \frac{227.08}{56.063} = 4.05 \text{ kips/in.}$$

B1.11.2 Required Nominal Resistance

$V_n = V_h / \varphi = 4.05 / 0.9 = 4.5$ kip / in.

B1.11.3 Required Interface Shear Reinforcement

The nominal shear resistance of the interface surface is:

$$V_n = cA_{cv} + \mu\left[A_{vf} f_y + P_c\right]$$ [LRFD Eq. 5.8.4.1-1]

c = cohesion factor [LRFD Art. 5.8.4.2]

μ = friction factor [LRFD Art. 5.8.4.2]

A_{cv} = area of concrete engaged in shear transfer, in.^{2}.

A_{vf} = area of shear reinforcement crossing the shear plane, in.^{2}

P_c = permanent net compressive force normal to the shear plane, kips

f_y = shear reinforcement yield strength, ksi

For concrete placed against clean, hardened concrete and free of laitance, but not an intentionally roughened surface: [LRFD Art. 5.8.4.2]

c = 0.075 ksi

$\mu = 0.6\lambda$, where λ = 1.0 for normal weight concrete, and therefore,

$\mu = 0.6$

The actual contact width, b_v, between the slab and the beam is 2(15.75) = 31.5 in.

A_{cv} = (31.5in.)(1in.) = 31.5 in.^{2}

The LRFD Eq. 5.8.4.1-1 can be solved for A_{vf} as follows:

$$4.5 = 0.075 \times 31.5 + 0.6\left[A_{vf}(60) + 0.0\right]$$

Solving for A_{vf} = 0.0594 in.^{2}/in. = 0.713 in.^{2} / ft.

Use 1 # 4 double legged. For the required A_{vf} = 0.713 in.^{2} / ft., the required spacing can be calculated as,

$$s = \frac{A_v \times 12}{A_{vf}} = \frac{0.392 \times 12}{0.713} = 6.6 \text{ in.}$$

Ultimate horizontal shear stress between slab and top of girder can be calculated,

$$V_{ult} = \frac{V_n \times 1000}{b_f} = \frac{4.5 \times 1000}{31.5} = 143.86 \text{ psi}$$

B.2.12 PRETENSIONED ANCHORAGE ZONE

[LRFD Art. 5.10.10]

B.2.12.1 Anchorage Zone Reinforcement

[LRFD Art. 5.10.10.1]

Design of the anchorage zone reinforcement is computed using the force in the strands just at transfer:

Force in the strands at transfer = F_{pi} = 64 (0.153)(202.5) = 1982.88 kips

The bursting resistance, P_r, should not be less than 4% of F_{pi}

$$P_r = f_s A_s \geq 0.04 F_{pi} = 0.04(1982.88) = 79.32 kips$$

Where

A_s = total area of vertical reinforcement located within a distance of h/4 from the end of the beam, in ^{2}.
f_s = stress in steel not exceeding 20 ksi.
Solving for required area of steel A_s= 79.32 /20 = 3.97 in.^{2}

Atleast 3.97 in.^{2} of vertical transverse reinforcement should be provided within a distance of (h/4 = 62 / 4 = 15.5 in.). from the end of the beam.
Use (7) #5 double leg bars at 2.0 in. spacing starting at 2 in. from the end of the beam. The provided A_s = 7(2)0.31 = 4.34 in.^{2} > 3.97 in.^{2} O.K.

B.2.12.2 Confinement Reinforcement

[LRFD Art. 5.10.10.2]

Transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.

B.2.13 DEFLECTION AND CAMBER

B.2.13.1 Maximum Camber Calculations Using Hyperbolic Functions Method

TxDOT's prestressed bridge design software, PSTRS 14 uses the Hyperbolic Functions Method proposed by Sinno Rauf, and Howard L Furr (1970) for the calculation of maximum camber. This design example illustrates the PSTRS 14 methodology for calculation of maximum camber.

Step1: Total Prestress after release

$$P = \frac{P_{si}}{\left(1 + p\,n + \dfrac{e_c^2 A_s\, n}{I}\right)} + \frac{M_D\, e_c\, A_s\, n}{I\left(1 + p\,n + \dfrac{e_c^2 A_s\, n}{I}\right)}$$

where,

P_{si} = total prestressing force = 1,811.295 kips

I = moment of inertia of non-composite section = 403,020 in.^{4}

e_c = eccentricity of pretensioning force at the midspan = 18.743 in.

M_D = Moment due to self-weight of the beam at midspan = 1,714.65 k-ft.

A_s = Area of strands = number of strands (area of each strand)

= 64(0.153) = 9.792 in.^{2}

$p = A_s / A_n$

where,

A_n = Area of cross-section of beam = 1120 in.^{2}

p = 9.972/1120 = 0.009

PSTRS14 uses final concrete strength to calculate E_c,

E_c = modulus of elasticity of the beam concrete, ksi

$$= 33(w_c)^{3/2}\sqrt{f'_c} = 33(150)^{1.5}\sqrt{5587}\ \frac{1}{1000} = 4{,}531.48 \text{ ksi}$$

E_{ps} = Modulus of elasticity of prestressing strands = 28,500 ksi

$n = E_{ps}/E_c$ = 28500/4531.48= 6.29

$$\left(1 + pn + \frac{e_c^2 A_s n}{I}\right) = 1 + (0.009)(6.29) + \frac{(18.743^2)(9.792)(6.29)}{403020} = 1.109$$

$$P = \frac{P_{si}}{\left(1 + p\,n + \dfrac{e_c^2 A_s\, n}{I}\right)} + \frac{M_D\, e_c\, A_s\, n}{I\left(1 + p\,n + \dfrac{e_c^2 A_s\, n}{I}\right)}$$

$$= \frac{1811.295}{1.109} + \frac{(1714.65)(12 \text{ in./ft.})(18.743)(9.792)(6.29)}{403020(1.109)}$$

= 1632.68 + 53.13 = 1,685.81 kips

Concrete Stress at steel level immediately after transfer

$$f_{ci}^{s} = P\left(\frac{1}{A} + \frac{e_c^{2}}{I}\right) - f_c^{s}$$

where,

f_c^s = Concrete stress at steel level due to dead loads

$$= \frac{M_D\, e_c}{I} = \frac{(1714.65)(12 \text{ in./ft.})(18.743)}{403020} = 0.957 \text{ ksi}$$

$$f_{ci}^{s} = 1685.81\left(\frac{1}{1120} + \frac{18.743^2}{403020}\right) - 0.957 = 2.018 \text{ ksi}$$

Step2: Ultimate time-dependent strain at steel level

$$\varepsilon_{c1}^{s} = \varepsilon_{cr}^{\infty} f_{ci}^{s} + \varepsilon_{sh}^{\infty}$$

where,

$\varepsilon_{cr}^{\infty}$ = ultimate unit creep strain = 0.00034 in./in. (this value is prescribed by Sinno et. al. (1970)

$\varepsilon_{sh}^{\infty}$ = ultimate unit creep strain = 0.000175 in./in. (this value is prescribed by Sinno et. al. (1970))

$$\varepsilon_{c1}^{\infty} = 0.00034(2.018) + 0.000175 = 0.0008611 \text{ in./in.}$$

Step3: Adjustment of total strain in step 2

$$\varepsilon_{c2}^{s} = \varepsilon_{c1}^{s} - \varepsilon_{c1}^{s}\, E_{ps}\frac{A_s}{E_{ci}}\left(\frac{1}{A_n} + \frac{e_c^{2}}{I}\right)$$

$$= 0.0008611 - 0.0008611\ (28500)\frac{9.792}{4531.48}\left(\frac{1}{1120} + \frac{18.743^2}{403020}\right)$$

= 0.000768 in./in.

Step4: Change in concrete stress at steel level

$$\Delta f_c^{s} = \varepsilon_{c2}^{s}\, E_{ps} A_s\left(\frac{1}{A_n} + \frac{e_c^{2}}{I}\right) = 0.000768\ (28500)(9.792)\left(\frac{1}{1120} + \frac{18.743^2}{403020}\right)$$

Δf_c^s = 0.375 ksi

Step5: Correction of the total strain from step2

$$\varepsilon^{s}_{c4} = \varepsilon^{\infty}_{cr} + \left(f^{s}_{ci} - \frac{\Delta f^{s}_{c}}{2} \right) + \varepsilon^{\infty}_{sh}$$

$$\varepsilon^{s}_{c4} = 0.00034 \left(2.018 - \frac{0.375}{2} \right) + 0.000175 = 0.0007974 \text{ in./in.}$$

Step6: Adjustment in total strain from step 5

$$\varepsilon^{s}_{c5} = \varepsilon^{s}_{c4} - \varepsilon^{s}_{c4}\, E_{ps} \frac{A_s}{E_c} \left(\frac{1}{A_n} + \frac{e_c^{2}}{I} \right)$$

$$= 0.0007974 - 0.0007974\ (28500)\ \frac{9.792}{4531.48} \left(\frac{1}{1120} + \frac{18.743^2}{403020} \right)$$

$$= 0.000711 \text{ in./in.}$$

Step 7: Change in concrete stress at steel level

$$\Delta f^{s}_{c1} = \varepsilon^{s}_{c5}\, E_{ps}\, A_s \left(\frac{1}{A_n} + \frac{e_c^{2}}{I} \right)$$

$$= 0.000711(28500)(9.792) \left(\frac{1}{1120} + \frac{18.743^2}{403020} \right)$$

$$\Delta f^{s}_{c1} = 0.350 \text{ ksi}$$

Step 8: Correction of the total strain from step 5

$$\varepsilon^{s}_{c6} = \varepsilon^{\infty}_{cr} + \left(f^{s}_{ci} - \frac{\Delta f^{s}_{c1}}{2} \right) + \varepsilon^{\infty}_{sh}$$

$$\varepsilon^{s}_{c6} = 0.00034 \left(2.018 - \frac{0.350}{2} \right) + 0.000175 = 0.000802 \text{ in./in.}$$

Step9: Adjustment in total strain from step 8

$$\varepsilon^{s}_{c7} = \varepsilon^{s}_{c6} - \varepsilon^{s}_{c6}\, E_{ps} \frac{A_s}{E_{ci}} \left(\frac{1}{A_n} + \frac{e_c^{2}}{I} \right)$$

$$= 0.000802 - 0.000802\ (28500)\ \frac{9.792}{4531.48} \left(\frac{1}{1120} + \frac{18.743^2}{403020} \right) = 0.000715 \text{ in./in.}$$

Step 10: Computation of initial prestress loss

$$PL_i = \frac{Psi\ -\ P}{Psi} = \frac{1811.295 - 1685.81}{1811.295} = 0.0693$$

Step 11: Computation of Final Prestress loss

$$PL^{\infty} = \frac{\varepsilon_{c7}^{\infty}\ E_{ps}\, A_s}{P_{si}} = \frac{0.000715(28500)(9.792)}{1811.295} = 0.109$$

Total Prestress loss

$$PL = PL_i + PL^{\infty} = 100(0.0693 + 0.109) = 17.83\%$$

Step 12: Initial deflection due to dead load

$$C_{DL} = \frac{5\ w\ L^4}{384\ E_c\, I}$$

where,

w = weight of beam = 1.167 kips/ft.

L = span length = 108.417 ft.

$$C_{DL} = \frac{5\left(\dfrac{1.167}{12\text{ in./ft.}}\right)[(108.417)(12\text{ in./ft.})]^4}{384(4531.48)(403020)} = 1.986\text{ in.}$$

Step 13: Initial Camber due to prestress

M/EI diagram is drawn for the moment caused by the initial prestressing, is shown in Figure B.2.9. Due to debonding of strands, the number of strands vary at each debonding section location. Strands that are bonded, achieve their effective prestress level at the end of transfer length. Points 1 through 6 show the end of transfer length for the preceding section. The *M/EI* values are calculated as,

$$\frac{M}{EI} = \frac{P_{si} \times ec}{E_c I}$$

The *M/EI* values are calculated for each point 1 through 6 and are shown in Table B.2.14. The initial Camber due to prestress, C_{pi}, can be calculated by Moment Area Method, by taking the moment of the *M/EI* diagram about the end of the beam.

C_{pi} = 3.88 in.

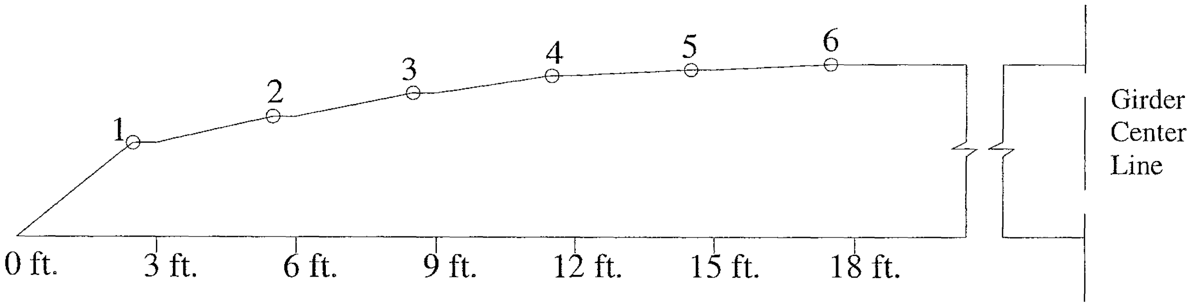

Figure B.2.9 M/EI Diagram to Calculate the Initial Camber due to Prestress

Table B.2.14 *M/EI* Values at the End of Transfer Length

Identifier for the End of Transfer Length	P_{si} **(kips)**	*ec* **(in.)**	*M/EI* **(in.^{3})**
1	1018.864	18.056	1.01E-05
2	1301.882	18.177	1.30E-05
3	1528.296	18.475	1.55E-05
4	1698.107	18.647	1.73E-05
5	1754.711	18.697	1.80E-05
6	1811.314	18.743	1.86E-05

Step 14: Initial Camber

$C_i = C_{pi} - C_{DL} = 3.88 - 1.986 = 1.894$ in.

Step 15: Ultimate Time Dependent Camber

Ultimate strain $\varepsilon_e^s = \dfrac{f_{ci}^s}{E_c}$ = 2.018/4531.48 = 0.000445 in./in.

Ultimate camber $$C_t = C_i\left(1 - PL^{\infty}\right)\frac{\varepsilon_{cr}^{\infty}\left(f_{ci}^s - \dfrac{\Delta f_{c1}^s}{2}\right) + \varepsilon_e^s}{\varepsilon_e^s}$$

$$= 1.894(1 - 0.109)\frac{0.00034\left(2.018 - \dfrac{0.347}{2}\right) + 0.000445}{0.000445}$$

C_t = 4.06 in. = 0.34 ft. ↑

B.2.13.2 Deflection Due to Beam Self-Weight

$$\Delta_{beam} = \frac{5 w_g L^4}{384 E_{ci} I}$$

where w_g = beam weight = 1.167 kips/ft.

Deflection due to beam self-weight at transfer

$$\Delta_{beam} = \frac{5(1.167/12)[(109.5)(12)]^4}{384(4262.75)(403020)} = 0.186 \text{ ft.} \downarrow$$

Deflection due to beam self-weight used to compute deflection at erection

$$\Delta_{beam} = \frac{5(1.167/12)[(108.417)(12)]^4}{384(4262.75)(403020)} = 0.165 \text{ ft.} \downarrow$$

B.2.13.3 Deflection Due to Slab and Diaphragm Weight

$$\Delta_{slab} = \frac{5 w_s L^4}{384 E_c I} + \frac{w_{dia} b}{24 E_c I}\left(3l^2 - 4b^2\right)$$

where,

w_s = slab weight = 1.15 kips/ft.

E_c = modulus of elasticity of beam concrete at service = 4529.45 ksi

$$\Delta_{slab} = \frac{5(1.15/12)[(108.417)(12)]^4}{384(4529.45)(403020)} + \frac{(3)(44.21\times 12)}{(24\times 4529.45\times 403020)}\left(3(108.417\times 12)^2 - 4(44.21\times 12)^2\right) = 0.163 \text{ ft.} \downarrow$$

B.2.13.4 Deflection Due to Superimposed Loads

$$\Delta_{SDL} = \frac{5 w_{SDL} L^4}{384 E_c I_c}$$

where,

w_{SDL} = superimposed dead load = 0.302 kips/ft.

I_c = moment of inertia of composite section = 1,054,905.38 in.^{4}

$$\Delta_{SDL} = \frac{5(0.302/12)[(108.417)(12)]^4}{384(4529.45)(1054905.38)} = 0.0155 \text{ ft.} \downarrow$$

Total deflection at service for all dead loads = 0.165 + 0.163 + 0.0155 = 0.34 ft.

B.2.13.5 Deflection Due to Live Load and Impact

The deflections due to live loads are not calculated in this example as they are not a design factor for TxDOT bridges.

B.2.14 COMPARISON OF RESULTS

In order to measure the level of accuracy in this detailed design example, the results are compared with that of PSTRS14 (TxDOT 2004). The summary of comparison is shown in Table B.2.15. In the service limit state design, the results of this example matches those of PSTRS14 with very insignificant differences. A difference up to 5.9 percent can be noticed for the top and bottom fiber stress calculation at transfer, and this is due to the difference in top fiber section modulus values and the number of debonded strands in the end zone, respectively. There is a huge difference of 24.5 percent in camber calculation, which can be due to the fact that PSTRS14 uses a single step hyperbolic functions method, whereas, a multi step approach is used in this detailed design example.

Table B.2.15 Comparison of Results for the AASHTO LRFD Specifications (PSTRS vs Detailed Design Example)

Design Parameters		**PSTRS14**	**Detailed Design Example**	**% diff. w.r.t. PSTRS14**
Prestress Losses, (%)	Initial	8.41	8.398	0.1
	Final	22.85	22.84	0.0
Required Concrete Strengths, (psi)	f'_{ci}	4,944	4,944	0.0
	f'_c	5,586	5,582	0.1
At Transfer (ends), (psi)	Top	-506	-533	-5.4
	Bottom	1,828	1,936	-5.9
At Service (midspan), (psi)	Top	2,860	2,856	0.1
	Bottom	-384	-383	0.3
Number of Strands		64	64	0.0
Number of Debonded Strands		(20+10)	(20+8)	2
M_u, (kip–ft.)		9,082	9,077	-0.1
ϕM_n, (kip–ft.)		11,888	12,028	-1.2
Ultimate Horizontal Shear Stress @ critical section, (psi)		143.3	143.9	0.0
Transverse Shear Stirrup (#4 bar) Spacing, (in.)		10.3	10	2.9
Maximum Camber, (ft.)		0.281	0.35	-24.6

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Appendix B

Detailed Examples for Interior Texas U54 Prestressed Concrete Bridge Girder Design

DRAFT

August 29, 2005

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B.1 Interior Texas U54 Prestressed Concrete Bridge Girder Design using AASHTO Standard Specifications

B.1.1 INTRODUCTION

Following is a detailed design example showing sample calculations for design of a typical Interior Texas prestressed precast concrete U54 beam supporting a single span bridge. The design is based on the *AASHTO Standard Specifications for Highway Bridges 17th Edition 2002*. The recommendations provided by the TxDOT Bridge Design Manual (TxDOT 2001) are considered in the design. The number of strands and concrete strength at release and at service are optimized using the TxDOT methodology.

B.1.2 DESIGN PARAMETERS

The bridge considered for design example has a span length of 110 ft. (c/c abutment distance), a total width of 46 ft. and total roadway width of 44 ft. The bridge superstructure consists of four Texas U54 beams spaced 11.5 ft. center-to-center designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck as shown in Figure B.1.1. The wearing surface thickness is 1.5 in., which includes the thickness of any future wearing surface. T501 type rails are considered in the design. AASHTO HS20 is the design live load. The relative humidity (RH) of 60% is considered in the design. The bridge cross-section is shown in Figure B.1.1.

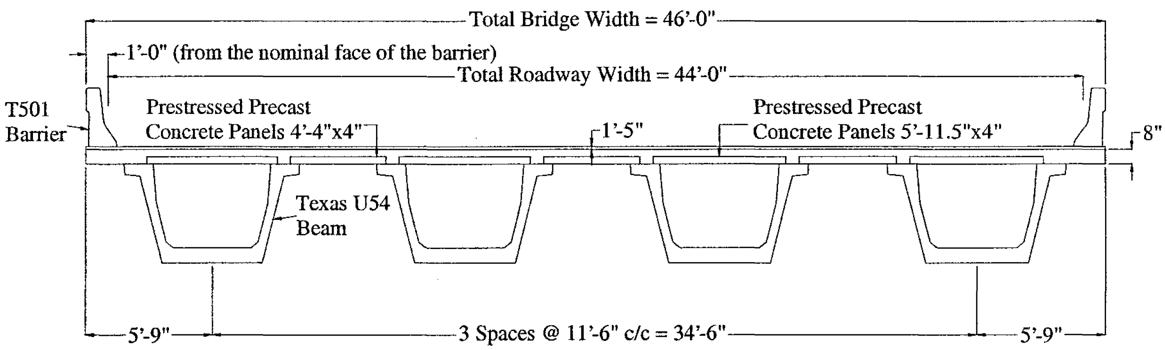

Figure B.1.1 Bridge Cross-Section Details

B.1.3 MATERIAL PROPERTIES

Cast-in-place slab:

Thickness, t_s = 8.0 in.

Concrete strength at 28-days, f_c' = 4,000 psi

Thickness of asphalt wearing surface (including any future wearing surfaces), t_w = 1.5 in.

Unit weight of concrete, w_c = 150 pcf

Precast beams: Texas U54 beam

Concrete strength at release, f_{ci}' = 4,000 psi*

Concrete strength at 28 days, $f_c' = 5{,}000$ psi*

Concrete unit weight, $w_c = 150$ pcf

*This value is taken as initial estimate and will be finalized based on most optimum design

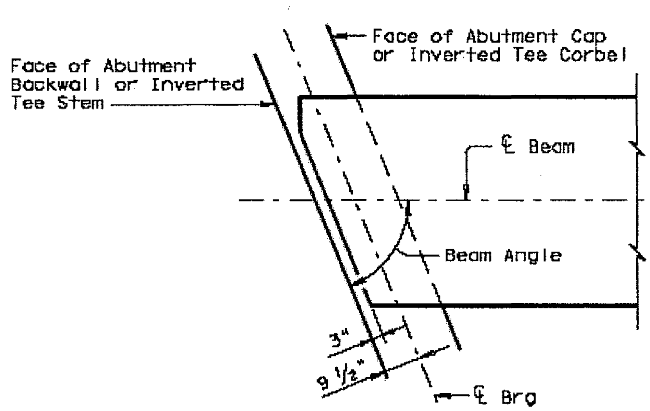

Figure B.1.2 Beam End Detail for Texas U54 Beams (TxDOT 2001)

Span length (c/c abutments) = 110 ft.–0 in.

Overall beam length = 110 ft. – 2(3 in.) = 109 ft.–6 in.

Design span = 110 ft. – 2(9.5 in.) = 108 ft.–5 in.

= 108.417 ft. (c/c of bearing)

Prestressing strands: ½ in. diameter, seven wire low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate stress, $f_s' = 270{,}000$ psi

Yield strength, $f_y = 0.9\ f_s' = 243{,}000$ psi [STD Art. 9.1.2]

Initial pretensioning, $f_{si} = 0.75\ f_s' = 202{,}500$ psi [STD Art. 9.15.1]

Modulus of elasticity, $E_s = 28{,}000$ ksi [STD Art. 9.16.2.1.2]

Non-prestressed reinforcement:

Yield strength, $f_y = 60{,}000$ psi

Unit weight of asphalt wearing surface = 140 pcf [TxDOT recommendation]

T501 type barrier weight = 326 plf per side

B.1.4 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

B.1.4.1 Non-Composite Section

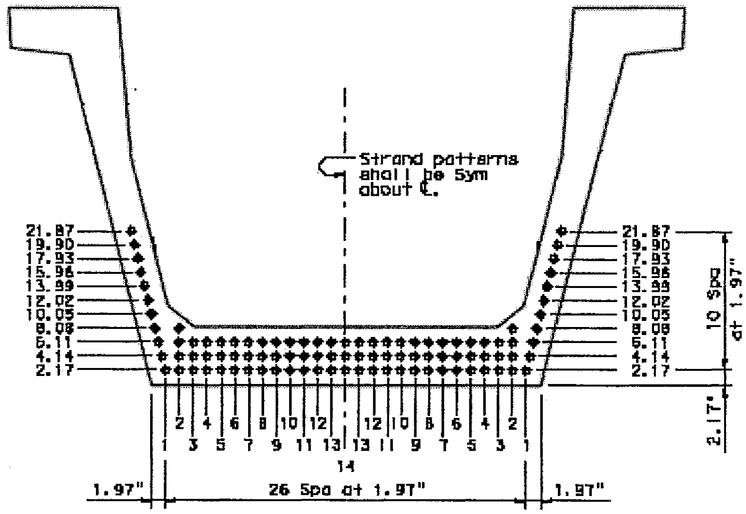

Figure B.1.3 Strand Pattern for Texas U54 Beams (TxDOT 2001)

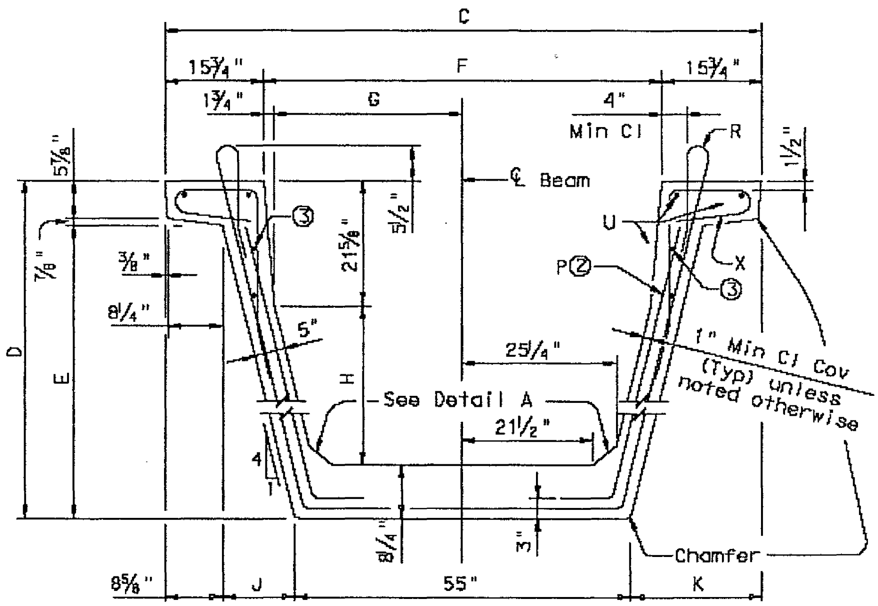

Figure B.1.4 Typical Section of Texas U54 Beams (TxDOT 2001)

Table B.1.1 Section Properties of Texas U54 beams (notations as used in Figure B.1.4, adapted from TxDOT Bridge Design Manual (TxDOT 2001))

C	D	E	F	G	H	J	K	Y_t	Y_b	Area	I	Weight
in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.^{2}	in.^{4}	plf
96	54	47.25	64.5	30.5	24.125	11.875	20.5	31.58	22.36	1,120	403,020	1,167

where,

I = moment of inertia about the centroid of the non-composite precast beam

Y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam

Y_t = distance from centroid to the extreme top fiber of the non-composite precast beam

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/Y_b = 403,020/22.36 = 18,024.15 in.^{3}

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/Y_t = 403,020/31.58 = 12,761.88 in.^{3}

B.1.4.2 Composite Section

B.1.4.2.1 Effective Flange Width

[STD Art. 9.8.3]

The Standard Specifications do not give specific guidelines regarding the calculation of effective flange width for open box sections. Following the LRFD recommendations, the effective flange width is determined as though each web is an individual supporting element. Thus, the effective flange width will be calculated according to guidelines of the Standard Specifications Art. 9.8.3 as below.

Effective web width of the precast beam is lesser of: [STD Art. 9.8.3.1]

b_e = top flange width = 15.75 in. (controls)

or, $b_e = 6\times$ (flange thickness) + web thickness + fillets

$= 6\times(5.875 \text{ in.} + 0.875 \text{ in.}) + 5.00 \text{ in.} + 0 \text{ in.} = 45.5 \text{ in.}$

The effective flange width is lesser of [STD Art. 9.8.3.2]

- 1/4 effective girder span length = $\dfrac{108.417 \text{ ft. } (12 \text{ in./ft.})}{4} = 325.25 \text{ in.}$
- $6\times$(Slab thickness on each side of the effective web width) + effective beam web width:

  $=6\times(8.0 \text{ in.}+8.0 \text{ in.}) + 15.75 \text{ in.} = 111.75 \text{ in.}$
- one-half the clear distance on each side of the effective web width plus the effective web width.

  $=0.5\times(4.0625 \text{ ft.}+ 4.8125 \text{ ft.}) + 1.3125 \text{ ft.} = 69 \text{ in.} = 5.75 \text{ ft.}$ (controls)

For the entire U-beam the effective flange width is $2\times(5.75 \text{ ft.}\times 12) = 138$ in.

$= 11.5$ ft.

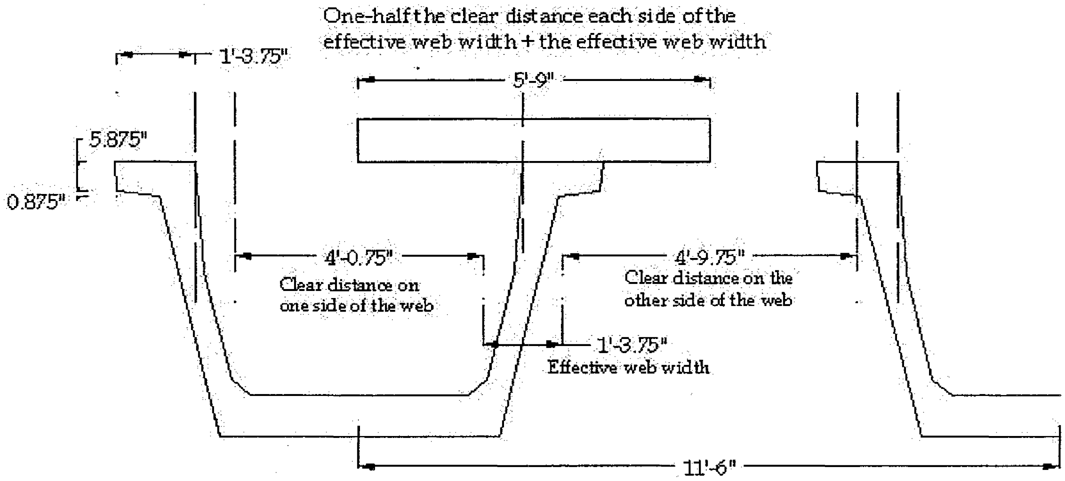

Figure B.1.5 Effective Flange Width Calculation

B.1.4.2.2 Modular Ratio Between Slab and Beam Material

Following the TxDOT Design recommendation the modular ratio between the slab and beam materials is taken as 1

$$n = \left(\frac{E_c \text{ for slab}}{E_c \text{ for beam}}\right) = 1$$

B.1.4.2.3 Transformed Section Properties

Transformed flange width = n×(effective flange width) = 1(138) = 138 in.

Transformed Flange Area = n×(effective flange width) (t_s) = 1(138)(8) = 1,104 in.^{2}

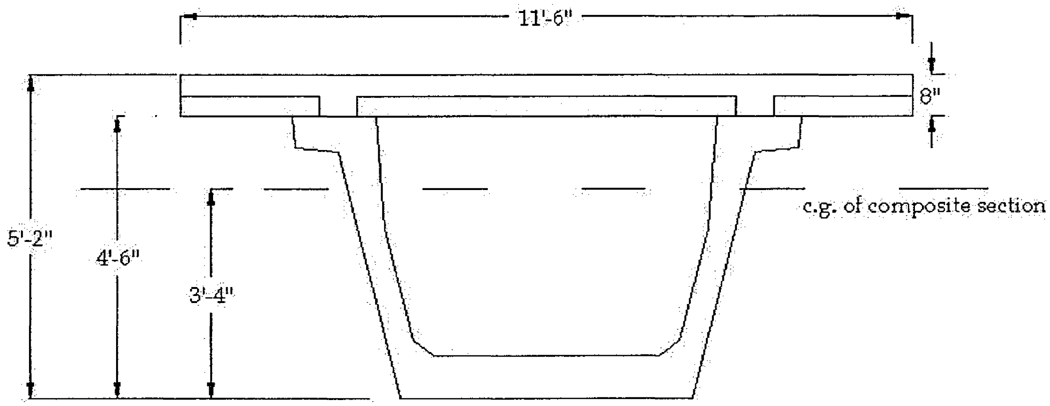

Figure B.1.6 Composite Section

Table B.1.2 Properties of Composite Section

	Transformed Area in.^{2}	y_b in.	A y_b in.	$A(y_{bc} - y_b)^2$	I in.^{4}	$I+A(y_{bc}- y_b)^2$ in.^{4}
Beam	1,120	22.36	25,043.2	350,488.43	403,020	753,508.43
Slab	1,104	58	64,032	355,711.56	5,888	361,599.56
∑	2,224		89,075.2			1,115,107.99

A_c = total area of composite section = 2,224 in.^{2}

h_c = total height of composite section = 62 in.

I_c = moment of inertia of composite section = 1,115,107.99 in.^{4}

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam = 89,075.2/ 2,224 = 40.05 in.

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam = 54 – 40.05 = 13.95 in.

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab = 62 – 40.05 = 21.95 in.

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam = I_c/y_{bc} = 1,115,107.99 / 40.05 = 27,842.9in.^{3}

S_{tg} = composite section modulus for top fiber of the precast beam = I_c/y_{tg} = 1,115,107.99 / 13.95 = 79,936.06 in.^{3}

S_{tc} = composite section modulus for top fiber of the slab = I_c/y_{tc} = 1,115,107.99 / 21.95 = 50,802.19 in.^{3}

B.1.5 SHEAR FORCES AND BENDING MOMENTS

B.1.5.1 Shear Forces and Bending Moments due to Dead Loads

B.1.5.1.1 Dead Loads

The self-weight of the beam and the weight of slab act on the non-composite simple span structure, while the weight of barriers, future wearing surface, and live load plus impact act on the composite simple span structure.

[STD Art. 3.3]

Self-weight of the beam = 1.167 kips/ft. [TxDOT Bridge Design Manual]

Weight of the CIP deck and precast panels on each beam

$$= (0.150 \text{ kcf})\left(\frac{8 \text{ in.}}{12 \text{ in./ft.}}\right)\left(\frac{138 \text{ in.}}{12 \text{ in./ft.}}\right)$$

$$= 1.15 \text{ kips/ft.}$$

Shear forces and bending moment values in the interior beam can be calculated by the following equations:

For $x = 0$ ft. – 44.21 ft.

$V_x = 3$ kips $\quad\quad M_x = 3x$ kips

For $x = 44.21$ ft. – 54.21 ft.

$V_x = 0$ kips $\quad\quad M_x = 3x - 3(x - 44.21)$ kips

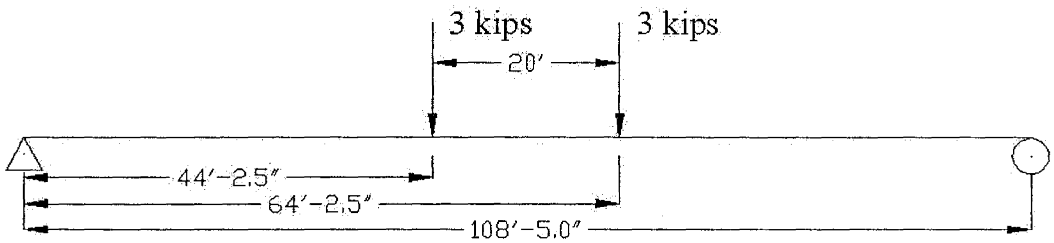

Figure B.1.7 Location of interior diaphragms on a simply supported bridge girder.

For U54 beam bridge design, TxDOT accounts for haunches in designs that require special geometry and where the haunch will be large enough to have a significant impact on the overall beam. Since this study is for typical bridges, a haunch will not be included for U54 beams for composite properties of the section and additional dead load considerations.

B.1.5.1.2 Superimposed Dead Load

TxDOT Design Manual recommends (Chap. 7 Sec. 24) that 1/3 of the rail dead load should be used for an interior beam adjacent to the exterior beam.

$$\text{Weight of T501 rails or barriers on each interior beam} = \left(\frac{326\text{ plf}/1000}{3}\right)$$

$$= 0.109\text{ kips/ft./interior beam}$$

The dead loads placed on the composite structure are distributed equally among all beams [STD Art. 3.23.2.3.1.1 & TxDOT Bridge Design Manual chap. 6 Sec. 3]

$$\text{Weight of 1.5 in. wearing surface} = \frac{(0.140\text{ pcf})\left(\frac{1.5\text{ in.}}{12\text{ in./ft.}}\right)(44\text{ ft.})}{4\text{ beams}} = 0.193\text{ kips/ft.}$$

Total superimposed dead load = 0.109 + 0.193 = 0.302 kip/ft.

B.1.5.1.3 Unfactored Shear Forces and Bending Moments

Shear forces and bending moments in the beam due to dead loads, superimposed dead loads at every tenth of the span and at critical sections (midspan and $h/2$) are shown in this section. The bending moment (M) and shear force (V) due to dead loads and super imposed dead loads at any section at a distance x are calculated using the following formulae.

$$M = 0.5wx\,(L - x)$$

$$V = w\,(0.5L - x)$$

Critical section for shear is located at a distance $h/2 = 62/2 = 31$ in. = 2.583 ft.

The shear forces and bending moments due to dead loads and superimposed dead loads are shown in Tables B.1.3 and B.1.4.

Table B.1.3 Shear forces due to Dead Loads

Distance	Section	Non-Composite Dead Load				Superimposed Dead Loads		Total Dead Load Shear Force
x	x/L	Beam Wt. V_g	Slab Wt. V_{slab}	Diaphram V_{dia}	Total $V_g+V_{slab}+V_{dia}$	Barrier Wt. V_b	Wearing Surface V_{ws}	
ft.		kips	kips	kips	kips	kips	kips	kips
0.000	0.000	63.26	62.34	3.00	128.60	5.91	10.46	144.97
2.583	0.024	60.25	59.37	3.00	122.62	5.63	9.96	138.21
10.842	0.100	50.61	49.87	3.00	103.48	4.73	8.37	116.58
21.683	0.200	37.96	37.40	3.00	78.36	3.55	6.28	88.19
32.525	0.300	25.30	24.94	3.00	53.24	2.36	4.18	59.78
43.367	0.400	12.65	12.47	3.00	28.12	1.18	2.09	31.39
54.209	0.500	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table B.1.4 Bending Moment due to Dead loads

Distance	Section	Non-Composite Dead Load				Superimposed Dead Loads		Total Dead Load Bending Moment
x	x/L	Beam Wt. M_g	Slab Wt. M_{slab}	Diaphram M_{dia}	Total $M_{slab} + M_{dia}$	Barrier Wt. M_b	Wearing Surface M_{ws}	
ft.		k – ft.	k – ft.	k – ft.	k – ft.	k – ft.	k – ft.	k – ft.
0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.583	0.024	159.51	157.19	7.75	324.45	14.90	26.38	365.73
10.842	0.100	617.29	608.30	32.53	640.83	57.66	102.09	1,417.87
21.683	0.200	1,097.36	1,081.38	65.05	1,146.43	102.50	181.48	2,527.77
32.525	0.300	1,440.30	1,419.32	97.58	1,516.90	134.53	238.20	3,329.93
43.367	0.400	1,646.07	1,622.09	130.10	1,752.19	153.75	272.23	3,824.24
54.209	0.500	1,714.65	1,689.67	132.63	1,822.30	160.15	283.57	3,980.67

B.1.5.2 Shear Forces and Bending Moments due to Live Load

B.1.5.2.1 Live Load

The AASHTO Standard Specifications requires the live load to be taken as either HS20 Standard truck loading or lane loading, whichever yields greater moments. The unfactored bending moments and Shear forces due to HS20 truck load are calculated using the following formulae given in the PCI Design manual (PCI 2003). [STD Art. 3.7.1.1]

For $x/L = 0 - 0.333$

Maximum unfactored bending moment, $M = \dfrac{72(x)[(L - x) - 9.33]}{L}$

For $x/L = 0.333 - 0.5$

Maximum unfactored bending moment, $M = \dfrac{72(x)[(L - x) - 4.67]}{L} - 112$

For $x/L = 0 - 0.5$

Maximum unfactored shear force, $V = \dfrac{72[(L - x) - 9.33]}{L}$

The bending moments and shear forces due to HS20 lane load are calculated using the following formulae

Maximum unfactored bending moment,

$$M = \frac{P(x)(L - x)}{L} + 0.5(w)(x)(L - x)$$

Maximum unfactored Shear Force, $V = \dfrac{Q(L - x)}{L} + (w)(\dfrac{L}{2} - x)$

where,

x = section at which bending moment or shear force is calculated

L = span length = 108.417 ft.

P = concentrated load for moment = 18 kips

Q = concentrated load for shear = 26 kips

w = uniform load per linear foot of load lane = 0.64 klf

Factored live load shear and bending moments are calculated by multiplying the distribution factor and the impact factor as follows

Factored bending moment M_{LL+I} = (bending moment per lane) (DF) (1+I)

Factored Shear Force V_{LL+I} = (shear force per lane) (DF) (1+I)

where,

DF is the Distribution factor

I is the Live load Impact factor

The shear forces and bending moments are shown in the Tables B.1.3 and B.1.4.

B.1.5.2.2 Live Load Distribution Factor for a Typical Interior Beam

As per TxDOT recommendation the live load distribution factor for moment for a precast prestressed concrete U54 interior beam is given by the following expression

$$DF_{mom} = \frac{S}{11} = \frac{11.5}{11} = 1.045 \text{ per truck/lane}$$ [TxDOT Chap.7 Sec 24]

where,

S = average interior beam spacing measured between beam center lines (ft.)

The minimum value of DF_{mom} is limited to 0.9.

For simplicity of calculation and because there is no significant difference, the distribution factor for moment is used also for shear as recommended by TxDOT Bridge Design Manual (Chap. 6 Sec-3)

[STD Art. 3.8]

B.1.5.2.3 Live Load Impact Factor

The live load impact factor is given by the following expression

$$I = \frac{50}{L + 125}$$ [STD Eq. 3-1]

where,

I = impact fraction to a maximum of 30%

L = Span length (ft.) = 108.417 ft. [STD Art. 3.8.2.2]

$$I = \frac{50}{108.417 + 125} = 0.214$$

Impact for shear varies along the span according to the location of the truck but the impact factor computed above is used for simplicity

Table B.1.5 Shear forces and Bending moments due to Live loads

Distance	Section	Live Load + Impact							
		HS 20 Truck Loading (controls)				HS20 Lane Loading			
x	x/L	Unfactored		Factored		Unfactored		Factored	
		Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment
ft.		kips	k-ft.	kips	k-ft.	kips	k-ft.	kips	k-ft.
0.000	0.000	65.80	0.00	83.52	0.00	34.69	0.00	36.27	0.00
2.583	0.024	64.09	165.54	81.34	210.10	33.06	87.48	34.56	91.45
10.842	0.100	58.60	635.38	74.38	806.41	28.10	338.53	29.38	353.92
21.683	0.200	51.40	1,114.60	65.24	1,414.62	22.20	601.81	23.21	629.16
32.525	0.300	44.20	1,437.73	56.10	1,824.74	17.00	789.88	17.77	825.78
43.370	0.400	37.00	1,626.98	46.96	2,064.93	12.49	902.73	13.06	943.76
54.210	0.500	29.80	1,671.37	37.83	2,121.27	8.67	940.34	9.07	983.08

B.1.5.3 Load Combinations

[STD Art. 3.22]

For service load design (Group I): 1.00 D + 1.00(L+I) [STD Table 3.22.1A]

where,

D = dead load

L = live load

I = Impact factor

For load factor design (Group I): 1.3[1.00D + 1.67(L+I)] [STD Table 3.22.1A]

B.1.6 ESTIMATION OF REQUIRED PRESTRESS

B.1.6.1 Service load Stresses at Midspan

The preliminary estimate of the required prestress and number of strands is based on the stresses at midspan

Bottom tensile stresses at midspan due to applied loads

$$f_b = \frac{M_g + M_S}{S_b} + \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

Top tensile stresses at midspan due to applied loads

$$f_t = \frac{M_g + M_S}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}}$$

where,

f_b = concrete stress at the bottom fiber of the beam

f_t = concrete stress at the top fiber of the beam

M_g = Unfactored bending moment due to beam self-weight

M_S = Unfactored bending moment due to slab, diaphragm weight

M_{SDL} = Unfactored bending moment due to super imposed dead load

M_{LL+I} = Factored bending moment due to super imposed dead load

Substituting the bending moments and section modulus values, bottom tensile stress at mid span is:

$$f_b = \frac{(1714.64 + 1689.66 + 132.63)(12)}{18024.15} + \frac{(443.72 + 2121.27)(12)}{27842.9} = 3.46 \text{ ksi}$$

$$f_t = \frac{(1714.64 + 1689.66 + 132.63)(12)}{12761.88} + \frac{(443.72 + 2121.27)(12)}{79936.06} = 3.71 \text{ ksi}$$

B.1.6.2 Allowable Stress Limit

At service load conditions, allowable tensile stress is

$$F_b = 6\sqrt{f_c'} = 6\sqrt{5000}\left(\frac{1}{1000}\right) = 0.424 \text{ ksi}$$ [STD Art. 9.15.2.2]

B.1.6.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – allowable tensile stress at final = $f_b - F_b$

= 3.46 – 0.424= 3.036 ksi

Assuming the distance from the center of gravity of strands to the bottom fiber of the beam is equal to y_{bs} = 2 in.

Strand eccentricity at midspan:

$$e_c = y_b - y_{bs} = 22.36 - 2 = 20.36 \text{ in.}$$

Bottom fiber stress due to prestress after losses:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}\, e_c}{S_b}$$

where,

P_{se} = effective pretension force after all losses

$$3.036 = \frac{P_{se}}{1120} + \frac{20.36\, P_{se}}{18024.15}$$

Solving for P_{se} we get,

P_{se} = 1,501.148 kips

Assuming final losses = 20% of f_{si}

Assumed final losses = 0.2(202.5 ksi) = 40.5 ksi

The prestress force per strand after losses

= (cross-sectional area of one strand) [f_{si} – losses]

= 0.153(202.5 – 40.5] = 24.786 kips

Number of strands required = 1500.159/24.786 = 60.56

Try 62 – ½ in. diameter, 270 ksi strands

Strand eccentricity at midspan after strand arrangement

$$e_c = 22.36 - \frac{27(2.17)+27(4.14)+8(6.11)}{62} = 18.934 \text{ in.}$$

P_{se} = 62(24.786) = 1,536.732 kips

$$f_b = \frac{1536.732}{1120} + \frac{18.934(1536.732)}{18024.15}$$

$$= 1.372 + 1.614 = 2.986 \text{ ksi} < f_b \text{ reqd.} = 3.034 \text{ ksi}$$

Try 64 – ½ in. diameter, 270 ksi strands

Strand eccentricity at midspan after strand arrangement

$$e_c = 22.36 - \frac{27(2.17)+27(4.14)+10(6.11)}{64} = 18.743 \text{ in.}$$

P_{se} = 64(24.786) = 1,586.304 kips

$$f_b = \frac{1586.304}{1120} + \frac{18.743(1586.304\ )}{18024.15}$$

$$= 1.416 + 1.650 = 3.066 \text{ ksi} > f_b \text{ reqd.} = 3.036 \text{ ksi}$$

Therefore, use 64 strands

Number of Strands	Distance from bottom (in.)
27	2.17
27	4.14
10	6.11

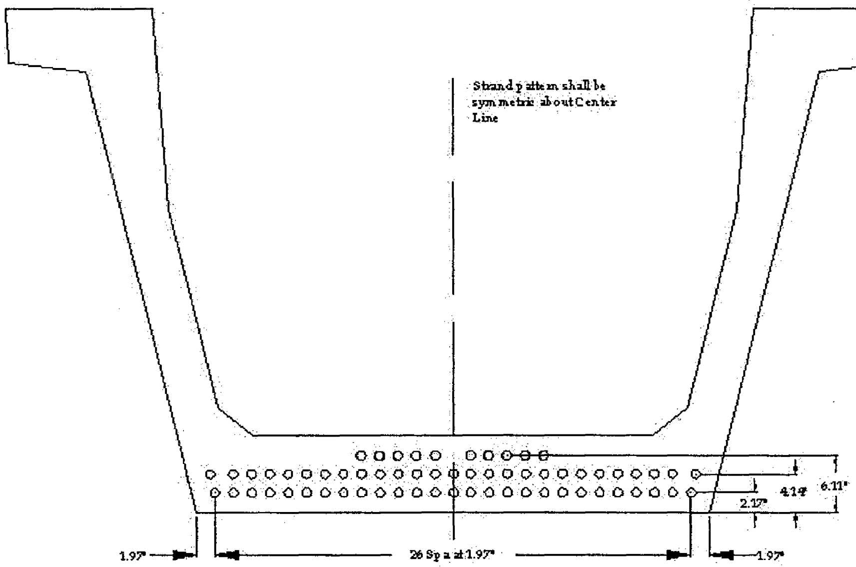

Fig. B.1.8 Initial Strand Pattern

B.1.7 PRESTRESS LOSSES

[STD Art. 9.16.2]

Total prestress losses = $SH + ES + CR_C + CR_S$ [STD Eq. 9-3]

where,

SH = loss of prestress due to concrete shrinkage

EC = loss of prestress due to elastic shortening

CR_C = loss of prestress due to creep of concrete

CR_S = loss of prestress due to relaxation of Prestressing steel

Number of strands = 64

A number of iterations will be performed to arrive at the optimum f_c' and f_{ci}'

B.1.7.1 Iteration 1

[STD Art. 9.16.2.1.1]

B.1.7.1.1 Shrinkage

[STD Eq. 9-4]

$SH = 17,000 - 150\ RH$

where, RH is the relative humidity = 60%

$$SH = [17000 - 150(60)]\frac{1}{1000} = 8 \text{ ksi}$$

B.1.7.1.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where,

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c^2}{I} - \frac{(M_g)e_c}{I}$$

f_{cir} = average concrete stress at the center of gravity of the prestressing steel due to pretensioning force and dead load of beam immediately after transfer

P_{si} = pretension force after allowing for the initial losses, assuming 8% initial losses = (number of strands)(area of each strand)[0.92(0.75 f_s')]

= 64(0.153)(0.92)(0.75)(270) = 1,824.25 kips

M_g = Unfactored bending moment due to beam self weight = 1714.64 k-ft.

e_c = eccentricity of the strand at the midspan = 18.743 in.

$$f_{cir} = \frac{1824.25}{1120} + \frac{1824.25\,(18.743)^2}{403020} - \frac{1714.64(12)(18.743)}{403020}$$

= 1.629 + 1.590 – 0.957 = 2.262 ksi

Assuming $f_{ci}' = 4{,}000$ psi

$$E_{ci} = (150)^{1.5}(33)\sqrt{4000}\ \frac{1}{1000} = 3{,}834.254 \text{ ksi}$$ [STD Eq. 9-8]

$$ES = \frac{28000}{3834.254}(2.262) = 16.518 \text{ ksi}$$

B.1.7.1.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

$CR_C = 12f_{cir} - 7f_{cds}$ [STD Eq. 9-9]

where,

f_{cds} = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the pretensioning force is applied

$$f_{cds} = \frac{M_S\,e_c}{I} + \frac{M_{SDL}\,(y_{bc} - y_{bs})}{I_c}$$

where,

M_S = slab + diaphragm = 1,822.29 k-ft.

M_{SDL} = superimposed dead load moment = 443.72 k-ft.

y_{bc} = 40.05 in.

y_{bs} = the distance from center of gravity of the strand at midspan to the bottom of the beam = 22.36 – 18.743 = 3.617 in.

I = moment of inertia of the non-composite section =403,020 in.^{4}

I_c = moment of inertia of composite section = 1,115,107.99 in.^{4}

$$f_{cds} = \frac{1822.29(12)(18.743)}{403020} + \frac{(443.72)(12)(40.05 - 3.617)}{1115107.99}$$

$$= 1.017 + 0.174 = 1.191 \text{ ksi}$$

$$CR_C = 12(2.262) - 7(1.191) = 18.807 \text{ ksi}$$

B.1.7.1.4 Relaxation of Prestressing Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strand

$CR_S = 5000 - 0.10\,ES - 0.05(SH + CR_C)$ [STD Eq. 9-10A]

$$= [5000 - 0.10(16518) - 0.05(8000 + 18807)]\left(\frac{1}{1000}\right) = 2.008\text{ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + 0.5CRs)100}{0.75f_s'}$$

$$= \frac{[16.518+0.5(2.008)]100}{0.75(270)} = 8.653\% > 8\% \text{ (assumed initial prestress losses)}$$

Therefore, next trial is required assuming 8.653% initial losses

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where,

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}\, e_c^2}{I} - \frac{(M_g)\, e_c}{I}$$

P_{si} = pretension force after allowing for the initial losses, assuming 8.653% initial losses = (number of strands)(area of each strand)[0.9135(0.75 f_s')]

= 64(0.153)(0.9135)(0.75)(270) = 1,811.3 kips

M_g = Unfactored bending moment due to beam self weight = 1,714.64 k-ft.

e_c = eccentricity of the strand at the midspan = 18.743 in.

$$f_{cir} = \frac{1811.3}{1120} + \frac{1811.3(18.743)^2}{403020} - \frac{1714.64(12)(18.743)}{403020}$$

$= 1.617 + 1.579 - 0.957 = 2.239$ ksi

Assuming $f'_{ci} = 4{,}000$ psi

$$E_{ci} = (150)^{1.5}(33)\sqrt{4000}\ \frac{1}{1000} = 3{,}834.254 \text{ ksi}$$ [STD Eq. 9-8]

$$ES = \frac{28000}{3834.254}(2.239) = 16.351 \text{ ksi}$$

$CR_C = 12f_{cir} - 7f_{cds}$

where,

f_{cds} will be same as calculated before

therefore, $f_{cds} = 1.191$

$CR_C = 12(2.239) - 7(1.191) = 18.531$ ksi.

For pretensioned members with 270 ksi low-relaxation strand

$CR_S = 5000 - 0.10\ ES - 0.05(SH + CR_C)$

$$= [5000 - 0.10(16351) - 0.05(8000 + 18531)]\left(\frac{1}{1000}\right) = 2.038 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + 0.5CRs)100}{0.75 f'_s}$$

$$= \frac{[16.351+0.5(2.038)]100}{0.75(270)} = 8.578\% < 8.653\% \text{ (assumed initial prestress losses)}$$

Therefore, next trial is required assuming 8.580% initial losses

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where,

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}\, e_c^2}{I} - \frac{(M_g)\, e_c}{I}$$

P_{si} = pretension force after allowing for the initial losses, assuming 8.580% initial losses = (number of strands)(area of each strand)[0.9142 (0.75 f'_s)]

$= 64(0.153)(0.9142)(0.75)(270) = 1{,}812.75$ kips

$$f_{cir} = \frac{1812.75}{1120} + \frac{1812.75(18.743)^2}{403020} - \frac{1714.64(12)(18.743)}{403020}$$

$= 1.619 + 1.580 - 0.957 = 2.242$ ksi

Assuming $f'_{ci} = 4{,}000$ psi

$$E_{ci} = (150)^{1.5}(33)\sqrt{4000}\ \frac{1}{1000} = 3{,}834.254 \text{ ksi}$$ [STD Eq. 9-8]

$$ES = \frac{28000}{3834.254}(2.242) = 16.372 \text{ ksi}$$

$CR_C = 12f_{cir} - 7f_{cds}$

where,

f_{cds} will be same as calculated before

therefore, $f_{cds} = 1.191$

$CR_C = 12(2.242) - 7(1.191) = 18.567$ ksi.

For pretensioned members with 270 ksi low-relaxation strand

$CR_S = 5000 - 0.10\,ES - 0.05(SH + CR_C)$

$$= [5000 - 0.10(16372) - 0.05(8000 + 18567)]\left(\frac{1}{1000}\right) = 2.034 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + 0.5CR_S)100}{0.75f'_s}$$

$$= \frac{[16.372 + 0.5(2.034)]100}{0.75(270)} = 8.587\% \approx 8.580\% \text{ (assumed initial prestress losses)}$$

B.1.7.1.5
Total Losses at Transfer

Total initial losses = $(ES + 0.5CR_S)$ = [16.372+0.5(2.034)] = 17.389 ksi

f_{si} = effective initial prestress = 202.5 – 17.389 = 185.111 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 64(0.153)(185.111) = 1,812.607 kips

B.1.7.1.6
Total Losses at Service Loads

$SH = 8$ ksi

$ES = 16.372$ ksi

$CR_C = 18.587$ ksi

$CR_S = 2.034$ ksi

Total final losses = 8 + 16.372 + 18.587 + 2.034 = 44.973 ksi

or $\dfrac{44.973(100)}{0.75(270)}$ =22.21%

f_{se} = effective final prestress = 0.75(270) – 44.973 = 157.527 ksi

P_{se} = 64(0.153)(157.527) = 1,542.504 kips

B.1.7.1.7 Final Stresses at Midspan

Final stress in the bottom fiber at midspan:

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,ec}{S_b} - f_b$$

$$f_{bf} = \frac{1542.504}{1120} + \frac{18.743(1542.504)}{18024.15} - 3.458$$

= 1.334 + 1.554 - 3.458 = -0.57 ksi > -0.424 ksi N.G.

Therefore, try 66 strands

$$e_c = 22.36 - \frac{27(2.17) + 27(4.14) + 12(6.11)}{66} = 18.67 \text{ in.}$$

P_{se} = 66(0.153)(157.527) = 1,590.708 kips

$$f_{bf} = \frac{1590.708}{1120} + \frac{18.67(1590.708)}{18024.15} - 3.458$$

= 1.42 + 1.648 - 3.458 = -0.39 ksi < -0.424 ksi O.K.

Therefore, use 66 strands

Final concrete stress at the top fiber of the beam at midspan,

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,ec}{S_t} + f_t = \frac{1590.708}{1120} - \frac{18.67(1590.708)}{12761.88} + 3.71$$

= 1.42 – 2.327 + 3.71 = 2. 803 ksi

B.1.7.1.8 Initial Stresses at End

Initial prestress

P_{si} = 66(0.153)(185.111) = 1,869.251 kips

Initial concrete stress at top fiber of the beam at girder end

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si}\,ec}{S_t} + \frac{M_g}{S_t}$$

where,

M_g = Moment due to beam self weight at girder end = 0 k-ft.

$$f_{ti} = \frac{1869.251}{1120} - \frac{18.67(1869.251)}{12761.88}$$

$$= 1.669 - 2.735 = -1.066 \text{ ksi}$$

Tension stress limit at transfer is $7.5\sqrt{f'_{ci}}$ [STD Art. 9.15.2.1]

Therefore, $f'_{ci\ \text{reqd.}} = \left(\frac{1066}{7.5}\right)^2 = 20{,}202$ psi

Initial concrete stress at bottom fiber of the beam at girder end

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si}\,e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1869.251}{1120} + \frac{18.67(1869.251)}{18024.15}$$

$$= 1.669 + 1.936 = 3.605 \text{ ksi}$$

Compression stress limit at transfer is $0.6\,f'_{ci}$ [STD Art. 9.15.2.1]

Therefore, $f'_{ci\ \text{reqd.}} = \frac{3605}{0.6} = 6{,}009$ psi

B.1.7.1.9 Debonding of Strands and Debonding Length

The calculation for initial stresses at the girder end show that preliminary estimate of $f'_{ci} = 4{,}000$ psi is not adequate to keep the tensile and compressive stresses at transfer within allowable stress limits as per STD Art. 9.15.2.1. Therefore, debonding of strands is required to keep the stresses within allowable stress limits.

In order to be consistent with the TxDOT design procedures, the debonding of strands is carried out in accordance with the procedure followed in PSTRS14 (TxDOT 2004).

Two strands are debonded at a time at each section located at uniform increments of 3 ft. along the span length, beginning at the end of the girder. The debonding is started at the end of the girder because due to relatively higher initial stresses at the end, greater number of strands are required to be debonded, and debonding requirement, in terms of number of strands, reduces as the section moves away from the end of the girder. In order to make the most efficient use of debonding due to greater eccentricities in the lower rows, the debonding at each section begins at the bottom most row and goes up. Debonding at a particular section will continue until the initial stresses are within the allowable stress limits or until a debonding limit is reached. When the debonding limit is reached, the initial concrete strength is increased and the design cycles to convergence. As per TxDOT Bridge Design Manual (TxDOT 2001) the limits of debonding for partially debonded strands are described as follows:

1. Maximum percentage of debonded strands per row and per section
    a. TxDOT Bridge Design Manual (TxDOT 2001) recommends a maximum percentage of debonded strands per row should not exceed 75%.
    b. TxDOT Bridge Design Manual (TxDOT 2001) recommends a maximum percentage of debonded strands per section should not exceed 75%.
2. Maximum Length of debonding
    a. TxDOT Bridge Design Manual (TxDOT 2001) recommends to use the maximum debonding length chosen to be lesser of the following:
        i. 15 ft.
        ii. 0.2 times the span length, or
        iii. half the span length minus the maximum development length as specified in the 1996 AASHTO Standard Specifications for Highway Bridges, Section 9.28.

B.1.7.1.10 Maximum Debonding Length

As per TxDOT Bridge Design Manual (TxDOT 2001), the maximum debonding length is the lesser of the following:

a. 15 ft.
b. 0.2 (L), or
c. 0.5 (L) - l_d

where, l_d is the development length calculated based on AASHTO STD Art. 9.28.1 as follows:

$$l_d \geq \left(f_{su}^* - \frac{2}{3} f_{se} \right) D \qquad \text{[STD Eq. 9.42]}$$

where,

l_d = development length (in.)

f_{se} = effective stress in the prestressing steel after losses

= 157.527 (ksi)

D = nominal strand diameter = 0.5 in.

f_{su}^* = average stress in the prestressing steel at the ultimate load (ksi)

$$f_{su}^* = f_s' \left[1 - \left(\frac{\gamma^*}{\beta_1} \right) \left(\frac{\rho^* f_s'}{f_c'} \right) \right] \qquad \text{[STD Eq. 9.17]}$$

where,

f_s' = ultimate stress of prestressing steel (ksi)

γ^* = factor type of prestressing steel

= 0.28 for low-relaxation steel

f_c' = compressive strength of concrete at 28 days (psi)

ρ^* $= \dfrac{A_s^*}{bd}$ = ratio of prestressing steel

$$= \frac{0.153 \times 66}{138 \times 8.67 \times 12} = 0.00033$$

β_1 = factor for concrete strength

$$\beta_1 = 0.85 - 0.05 \frac{(f_c' \text{ - } 4000)}{1000} \qquad \text{[STD Art. 8.16.2.7]}$$

$$= 0.85 - 0.05 \frac{(5000 \text{ - } 4000)}{1000} = 0.80$$

$$f_{su}^* = 270 \left[1 - \left(\frac{0.28}{0.80} \right) \left(\frac{0.00033 \times 270}{5} \right) \right] = 268.32 \text{ ksi}$$

The development length is calculated as,

$$l_d \geq \left(268.32 - \frac{2}{3} 157.527 \right) \times 0.5$$

l_d = 6.8 ft.

As per STD Art. 9.28.3, the development length calculated above should be doubled.

l_d = 13.6 ft.

Hence, the debonding length is the lesser of the following,

a. 15 ft.

b. 0.2 × 108.417 = 21.68 ft.

c. 0.5 × 108.417 - 13.6 = 40.6 ft.

Hence, the maximum debonding length to which the strands can be debonded is 15 ft.

Table B.1.6 Calculation of Initial Stresses at Extreme Fibers and Corresponding Required Initial Concrete Strengths

	Location of the Debonding Section (ft. from end)						
	End	3	6	9	12	15	Midspan
Strands in Row No. 1 (bottom row)	27	27	27	27	27	27	27
Strands in Row No. 2	27	27	27	27	27	27	27
Strands in Row No. 3	12	12	12	12	12	12	12
Total No. of Strands at a Section	66	66	66	66	66	66	66
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1,869.25	1,869.25	1,869.25	1,869.25	1,869.25	1,869.25	1,869.25
ec (in.)	18.67	18.67	18.67	18.67	18.67	18.67	18.67
Top Fiber Stresses (ksi)	-1.066	-0.892	-0.728	-0.575	-0.431	-0.297	0.547
Corresponding $f'_{ci\ reqd}$ (psi)	20,202	14,145	9,422	5,878	3,302	1,568	912
Bottom Fiber Stresses (ksi)	3.605	3.482	3.366	3.258	3.156	3.061	2.464
Corresponding $f'_{ci\ reqd}$ (psi)	6,009	5,804	5,611	5,429	5,260	5,101	4,106

In Table B.1.6, the calculation of initial stresses at the extreme fibers and corresponding requirement of f'_{ci} suggests that the preliminary estimate of f'_{ci} to be 4,000 psi is inadequate. Since strand can not be debonded beyond the section located at 15 ft. from the end of the beam, so, f'_{ci} is increased from 4,000 psi to 5,101 psi and at all other section, where debonding can be done, the strands are debonded to bring the required f'_{ci} below 5,101 psi. Table B.1.7 shows the debonding schedule based on the procedure described earlier.

Table B.1.7 Debonding of Strands at Each Section

	Location of the Debonding Section (ft. from end)						
	End	3	6	9	12	15	Midspan
Row No. 1 (bottom row)	7	7	15	23	25	27	27
Row No. 2	17	27	27	27	27	27	27
Row No. 3	12	12	12	12	12	12	12
No. of Strands	36	46	54	62	64	66	66
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1,019.59	1,302.81	1,529.39	1,755.96	1,812.61	1,869.25	1,869.25
ec (in.)	17.95	18.01	18.33	18.57	18.62	18.67	18.67
Top Fiber Stresses (ksi)	-0.524	-0.502	-0.494	-0.496	-0.391	-0.297	0.547
Corresponding $f'_{ci\ reqd}$ (psi)	4,881	4,480	4,338	4,374	2,718	1,568	912
Bottom Fiber Stresses (ksi)	1.926	2.342	2.682	3.029	3.041	3.061	2.464
Corresponding $f'_{ci\ reqd}$ (psi)	3,210	3,904	4,470	5,049	5,069	5,101	4,106

B.1.7.2 Iteration 2

Following the procedure in iteration 1 another iteration is required to calculate prestress losses based on the new value of f'_{ci} = 5,101 psi. The results of this second iteration are shown in Table B.1.8

Table B.1.8 Results of Iteration No. 2

	Trial #1	Trial # 2	Trial # 3	Units
No. of Strands	66	66	66	
ec	18.67	18.67	18.67	in.
SR	8	8	8	ksi
Assumed Initial Prestress Loss	8.587	7.967	8.031	%
P_{si}	1,869.19	1,881.87	1,880.64	kips
M_g	1,714.65	1,714.65	1,714.65	k - ft.
f_{cir}	2.332	2.354	2.352	ksi
f_{ci}	5,101	5,101	5,101	psi
E_{ci}	4,329.91	4,329.91	4,329.91	ksi
ES	15.08	15.22	15.21	ksi
f_{cds}	1.187	1.187	1.187	ksi
CRc	19.68	19.94	19.92	ksi
CRs	2.11	2.08	2.08	ksi
Calculated Initial Prestress Loss	7.967	8.031	8.025	%
Total Prestress Loss	44.86	45.24	45.21	ksi

B.1.7.2.1 Total Losses at Transfer

Total Initial losses = $(ES + 0.5CRs)$ = [15.21 + 0.5(2.08)]= 16.25 ksi

f_{si} = effective initial prestress = 202.5 – 16.25 = 186.248 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 66(0.153)(186.248) = 1,880.732 kips

B.1.7.2.2 Total Losses at Service Loads

$SH = 8$ ksi

$ES = 15.21$ ksi

$CR_C = 19.92$ ksi

$CR_S = 2.08$ ksi

Total final losses = 8 + 15.21 + 19.92 + 2.08 = 45.21 ksi

or $\dfrac{45.21(100)}{0.75(270)}$ =22.32%

f_{se} = effective final prestress = 0.75(270) – 45.21 = 157.29 ksi

P_{se} = 66(0.153)(157.29) = 1,588.34 kips

B.1.7.2.3 Final Stresses at Midspan

Top fiber stress in concrete at midspan at service loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1588.34}{1120} - \frac{18.67(1588.34)}{12761.88} + 3.71$$

$$= 1.418 - 2.323 + 3.71 = 2.805 \text{ ksi}$$

Allowable compression stress limit for all load combinations = $0.6\,f'_c$

$f'_{c\,reqd}$ = 2805/0.6 = 4,675 psi [STD Art. 9.15.2.2]

Top fiber stress in concrete at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

$$= \frac{1588.34}{1120} - \frac{18.67(1588.34)}{12761.88} + \frac{(1714.64 + 1822.29)(12)}{12761.88} + \frac{443.72(12)}{79936.06}$$

$$= 1.418 - 2.323 + 3.326 + 0.067 = 2.49 \text{ ksi}$$

Allowable compression stress limit for effective pretension force

+ permanent dead loads = $0.4\,f'_c$ [STD Art. 9.15.2.2]

$f'_{c\,reqd}$ = 2490/0.4 = 6,225 psi (controls)

Top fiber stress in concrete at midspan due to live load

+ ½(effective prestress + dead loads)

$$f_{tf} = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}\right)$$

$$= \frac{2121.27(12)}{79936.06} + 0.5\left(\frac{1588.34}{1120} - \frac{18.67(1588.34)}{12761.88} + \frac{(1714.64 + 1822.29)(12)}{12761.88} + \frac{443.72(12)}{79936.06}\right)$$

$$= 0.318 + 0.5(1.418 - 2.323 + 3.326 + 0.067) = 1.629 \text{ ksi}$$

Allowable compression stress limit for effective pretension force

+ permanent dead loads = $0.4\, f_c'$ [STD Art. 9.15.2.2]

$f'_{c\ reqd} = 1562/0.4 = 3{,}905$ psi

Bottom fiber stress in concrete at midspan at service load

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\, e_c}{S_b} - f_b$$

$$f_{bf} = \frac{1588.34}{1120} + \frac{18.67(1588.34)}{18024.15} - 3.46$$

$$= 1.418 + 1.633 - 3.46 = -0.397 \text{ ksi}$$

Allowable tension in concrete = $6\sqrt{f_c'}$ [STD Art. 9.15.2.2]

$$f'_{c\ reqd.} = \left(\frac{3970}{6}\right)^2 = 4{,}366 \text{ psi}$$

B.1.7.2.4 Initial Stresses at Debonding Locations

With the same number of debonded strands, as was determined in the previous iteration, the top and bottom fiber stresses with their corresponding initial concrete strengths are calculated. It can be observed that at 15 ft. location, the f_{ci}' value is updated to 5,138 psi.

Table B.1.9 Debonding of Strands at Each Section

	Location of the Debonding Section (ft. from end)						
	0	3	6	9	12	15	54.2
Row No. 1 (bottom row)	7	7	15	23	25	27	27
Row No. 2	17	27	27	27	27	27	27
Row No. 3	12	12	12	12	12	12	12
No. of Strands	36	46	54	62	64	66	66
M_g (k-ft.)	0	185	359	522	675	818	1715
P_{si} (kips)	1,025.85	1,310.81	1,538.78	1,766.75	1,823.74	1,880.73	1,880.73
ec (in.)	17.95	18.01	18.33	18.57	18.62	18.67	18.67
Top Fiber Stresses (ksi)	-0.527	-0.506	-0.499	-0.502	-0.398	-0.303	0.540
Corresponding $f'_{ci\ reqd}$ (psi)	4,937	4,552	4,427	4,480	2,816	1,632	900
Bottom Fiber Stresses (ksi)	1.938	2.357	2.700	3.050	3.063	3.083	2.486
Corresponding $f'_{ci\ reqd}$ (psi)	3,229	3,929	4,500	5,084	5,105	5,138	4,143

C1.7.3 Iteration 3

Following the procedure in iteration 1, a third iteration is required to calculate prestress losses based on the new value of f'_{ci} = 5,138 psi. The results of this second iteration are shown in Table C1.7.2.2

Table B.1.10 Results of Iteration No. 3

	Trial #1	Trial # 2	Units
No. of Strands	66	66	
ec	18.67	18.67	in.
SR	8	8	ksi
Assumed Initial Prestress Loss	8.025	8.000	%
P_{si}	1,880.85	1,881.26	kips
M_g	1,714.65	1,714.65	k - ft.
f_{cir}	2.352	2.354	ksi
f_{ci}	5,138	5,138	psi
E_{ci}	4,346	4,346	ksi
ES	15.16	15.17	ksi
f_{cds}	1.187	1.187	ksi
CRc	19.92	19.94	ksi
CRs	2.09	2.09	ksi
Calculated Initial Prestress Loss	8.000	8.005	%
Total Prestress Loss	45.16	45.19	ksi

B.1.7.3.1 Total Losses at Transfer

Total initial losses = $(ES + 0.5CRs)$ = [15.17+0.5(2.09)]= 16.211 ksi

f_{si} = effective initial prestress = 202.5 – 16.211 = 186.289 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 66(0.153)(186.289) = 1,881.146 kips

B.1.7.3.2 Total Losses at Service Loads

SH = 8 ksi

ES = 15.17 ksi

CR_C = 19.94 ksi

CR_S = 2.09 ksi

Total final losses = 8 + 15.17 + 19.94 + 2.09 = 45.193 ksi

or $\dfrac{45.193\ (100)}{0.75(270)}$ =22.32%

f_{se} = effective final prestress = 0.75(270) – 45.193 = 157.307 ksi

P_{se} = 66(0.153)(157.307) = 1,588.486 kips

B.1.7.3.3 Final Stresses at Midspan

Top fiber stress in concrete at midspan at service loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + f_t = \frac{1588.486}{1120} - \frac{18.67(1588.486)}{12761.88} + 3.71$$

$$= 1.418 - 2.323 + 3.71 = 2.805 \text{ ksi}$$

Allowable compression stress limit for all load combinations = 0.6 f'_c

$f'_{c\ reqd.} = 2805/0.6 = 4{,}675$ psi [STD Art. 9.15.2.2]

Top fiber stress in concrete at midspan due to effective prestress + permanent dead loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

$$= \frac{1588.486}{1120} - \frac{18.67(1588.486)}{12761.88} + \frac{(1714.64 + 1822.29)(12)}{12761.88} + \frac{443.72(12)}{79936.06}$$

$$= 1.418 - 2.323 + 3.326 + 0.067 = 2.49 \text{ ksi}$$

Allowable compression stress limit for effective pretension force

+ permanent dead loads = 0.4 f'_c [STD Art. 9.15.2.2]

$f'_{c\ reqd} = 2490/0.4 = 6{,}225$ psi (controls)

Top fiber stress in concrete at midspan due to live load

+ ½(effective prestress + dead loads)

$$f_{tf} = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}\right)$$

$$= \frac{2121.27(12)}{79936.06} + 0.5\left(\frac{1588.486}{1120} - \frac{18.67(1588.486)}{12761.88} + \frac{(1714.64 + 1822.29)(12)}{12761.88} + \frac{443.72(12)}{79936.06}\right)$$

$$= 0.318 + 0.5(1.418 - 2.323 + 3.326 + 0.067) = 1.562 \text{ ksi}$$

Allowable compression stress limit for effective pretension force

+ permanent dead loads = 0.4 f'_c [STD Art. 9.15.2.2]

$f'_{c\ reqd.} = 1562/0.4 = 3{,}905$ psi

Bottom fiber stress in concrete at midspan at service load

$$f_{bf} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - f_b$$

$$f_{bf} = \frac{1588.486}{1120} + \frac{18.67(1588.486)}{18024.15} - 3.458$$

$= 1.418 + 1.645 - 3.46 = -0.397$ ksi

Allowable tension in concrete = $6\sqrt{f'_c}$ [STD Art. 9.15.2.2]

$$f'_{c\ reqd.} = \left(\frac{3970}{6}\right)^2 = 4{,}366 \text{ psi}$$

B.1.7.3.4 Initial Stresses at Debonding Location

With the same number of debonded strands, as was determined in the previous iteration, the top and bottom fiber stresses with their corresponding initial concrete strengths are calculated. It can be observed that at 15 ft. location, the f'_{ci} value is updated to 5,140 psi.

Table B.1.11 Debonding of Strands at Each Section

	Location of the Debonding Section (ft. from end)						
	0	3	6	9	12	15	54.2
Row No. 1 (bottom row)	7	7	15	23	25	27	27
Row No. 2	17	27	27	27	27	27	27
Row No. 3	12	12	12	12	12	12	12
No. of Strands	36	46	54	62	64	66	66
M_g (k-ft.)	0	185	359	522	675	818	1,715
P_{si} (kips)	1,026.08	1,311.10	1,539.12	1,767.14	1,824.14	1,881.15	1,881.15
ec (in.)	17.95	18.01	18.33	18.57	18.62	18.67	18.67
Top Fiber Stresses (ksi)	-0.527	-0.506	-0.499	-0.503	-0.398	-0.304	0.540
Corresponding $f'_{ci\ reqd}$ (psi)	4,937	4,552	4,427	4,498	2,816	1,643	900
Bottom Fiber Stresses (ksi)	1.938	2.358	2.701	3.051	3.064	3.084	2.487
Corresponding $f'_{ci\ reqd}$ (psi)	3,230	3,930	4,501	5,085	5,106	5,140	4,144

Since actual initial losses are 8.005% as compared to previously assumed 8.0% and f'_{ci} = 5,140 psi as compared to previously calculated f'_{ci} = 5,138 psi. These values are close enough, so no further iteration will be required. The optimized value of f'_c required is 6,225 psi. AASHTO Standard article 9.23 requires f'_{ci} to be atleast 4,000 for pretensioned members.

Use f'_c = 6,225 psi and f'_{ci} = 5,140 psi.

B.1.8

STRESS SUMMARY

B.1.8.1

Concrete Stresses at Transfer

B.1.8.1.1

Allowable Stress Limits

[STD Art. 9.15.2.1]

Compression: 0.6 f'_{ci} = 0.6(5140) = +3,084 psi = 3.084 ksi (compression)

Tension: The maximum allowable tensile stress is smaller of

$3\sqrt{f'_{ci}} = 3\sqrt{5140} = 215.1$ psi and 200 psi (controls)

$7.5\sqrt{f'_{ci}} = 7.5\sqrt{5140} = 537.71$ psi (tension) > 200 psi, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section to allow 537.71 ksi tensile stress in concrete.

B.1.8.1.2

Stresses at Beam End and at Transfer Length Section

Stresses at beam end and transfer length section need only be checked at release, because losses with time will reduce the concrete stresses making them less critical.

B.1.8.1.2.1

Stresses at Transfer Length Section

Transfer length = 50 (strand diameter)
= 50 (0.5) = 25 in. = 2.083 ft. [STD Art. 9.20.2.4]

Transfer length section is located at a distance of 2.083 ft. from end of the beam. Overall beam length of 109.5 ft. is considered for the calculation of bending moment at transfer length. As shown in Table B.1.11, the number of strands at this location, after debonding of strands, is 36.

Moment due to beam self weight, M_g = 0.5(1.167)(2.083)(109.5 – 2.083)
= 130.558 k –ft.

Concrete stress at top fiber of the beam

$$f_t = \frac{P_{si}}{A} - \frac{P_{si}\,e_t}{S_t} + \frac{M_g}{S_t}$$

P_{si} = 36(0.153)(185.946) = 1024.19 kips

Strand eccentricity at transfer section, e_c = 17.95 in.

$$f_t = \frac{1024.19}{1120} - \frac{17.95\,(1024.19)}{12761.88} + \frac{130.558(12)}{12761.88} = 0.915 - 1.44 + 0.123 = -0.403 \text{ ksi}$$

Allowable tension (with bonded reinforcement) = 537.71 psi > 403 psi (O.K.)

Compute stress limit for concrete at the bottom fiber of the beam

Concrete stress at the bottom fiber of the beam

$$f_b = \frac{P_{si}}{A} + \frac{P_{si}\, e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1024.19}{1120} + \frac{17.95\,(1024.19)}{18024.15} - \frac{130.558(12)}{18024.15} = 0.915 + 1.02 - 0.087 = 1.848 \text{ ksi}$$

Allowable compression = 3.084 ksi < 1.848 ksi (reqd.) (O.K.)

B.1.8.1.2.2
Stresses at Beam End

And the strand eccentricity at end of beam is:

$$e_c = 22.36 - \frac{7(2.17)+17(4.14)+12(6.11)}{36} = 17.95 \text{ in.}$$

P_{si} = 36 (0.153) (185.946) = 1024.19 kips

Concrete stress at the top fiber of the beam

$$f_t = \frac{1024.19}{1120} - \frac{17.95\,(1024.19)}{12761.88} = 0.915 - 1.44 = -0.526 \text{ ksi}$$

Allowable tension (with bonded reinforcement) = 537.71 psi > 526 psi (O.K.)

Concrete stress at the bottom fiber of the beam

$$f_b = \frac{P_{si}}{A} + \frac{P_{si}\, e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_b = \frac{1021.701}{1120} + \frac{17.95\,(1021.701)}{18024.15} = 0.915 + 1.02 = 1.935 \text{ ksi}$$

Allowable compression = 3.084 ksi > 1.935 ksi (reqd.) (O.K.)

B.1.8.1.3
Stresses at Midspan

Bending moment at midspan due to beam self –weight based on overall length

M_g = 0.5(1.167)(54.21)(109.5 – 54.21) = 1748.908 k-ft.

Concrete stress at top fiber of the beam at midspan

$$f_t = \frac{P_{si}}{A} - \frac{P_{si}\, e_c}{S_t} + \frac{M_g}{S_t}$$

$$f_t = \frac{1881.15}{1120} - \frac{17.95\,(1881.15)}{12761.88} + \frac{1748.908\,(12)}{12761.88} = 1.68 - 2.64 + 1.644 = 0.684 \text{ ksi}$$

Allowable compression: 3.084 ksi >> 0.684 ksi (reqd.) (O.K.)

Concrete stresses in bottom fibers of the beam at midspan

$$f_b = \frac{P_{si}}{A} + \frac{P_{si}\, e_c}{S_b'} - \frac{M_g}{S_b}$$

$$f_b = \frac{1881.15}{1120} + \frac{17.95(1881.15)}{18024.15} - \frac{1748.908(12)}{18024.15} = 1.68 + 1.87 - 1.164 = 2.386 \text{ ksi}$$

Allowable compression: 3.084 ksi > 2.386 ksi (reqd.) (O.K.)

B.1.8.1.4
Stress Summary at Transfer

	Top of beam f_t (ksi)	Bottom of beam f_b (ksi)
At End	-0.526	+1.935
At transfer length section from End	-0.403	+1.848
At Midspan	+0.684	+2.386

B.1.8.2
Concrete Stresses at Service Loads

B.1.8.2.1
Allowable Stress Limits

[STD Art. 9.15.2.2]

Compression

Case (I): for all load combinations

$0.60\, f_c' = 0.60(6225)/1000 = +3.74$ ksi (for precast beam)

$0.60\, f_c' = 0.60(4000)/1000 = +2.4$ ksi (for slab)

Case (II): for effective pretension force + permanent dead loads

$0.40\, f_c' = 0.40(6225)/1000 = +2.493$ ksi (for precast beam)

$0.40\, f_c' = 0.40(4000)/1000 = +1.6$ ksi (for slab)

Case (III): for live load +1/2(effective pretension force + dead loads)

$0.40\, f_c' = 0.40(6225)/1000 = +2.493$ ksi (for precast beam)

$0.40\, f_c' = 0.40(4000)/1000 = +1.6$ ksi (for slab)

Tension: $6\sqrt{f_c'} = 6\sqrt{6225}\left(\frac{1}{1000}\right) = -0.4737$ ksi

B.1.8.2.2
Stresses at Midspan

$P_{se} = 66(0.153)(157.307) = 1{,}588.49$ kips

Concrete stresses at top fiber of the beam at service loads

$$f_t = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}}$$

Case (I):

$$f_t = \frac{1588.49}{1120} - \frac{18.67(1588.49)}{12761.88} + \frac{(1714.64+1822.29)(12)}{12761.88} + \frac{(443.72+2121.278)(12)}{79936.06}$$

$f_t = 1.418 - 2.323 + 3.326 + 0.385 = 2.805$ ksi

Allowable compression: +3.84 ksi > +2.805 ksi (reqd.) (O.K.)

Case (II): Effective pretension force + permanent dead loads

$$f_t = \frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

$$f_t = \frac{1588.49}{1120} - \frac{18.67(1588.49)}{12761.88} + \frac{(1714.64+1822.29)(12)}{12761.88} + \frac{(443.72)(12)}{79936.06}$$

$f_t = 1.418 - 2.323 + 3.326 + 0.067 = 2.49$ ksi

Allowable compression: +2.493 ksi > +2.49 ksi (reqd.) (O.K.)

Case (III): Live load + ½(Pretensioning force + dead loads)

$$f_t = \frac{M_{LL+I}}{S_{tg}} + 0.5\left(\frac{P_{se}}{A} - \frac{P_{se}\,e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) =$$

$$\frac{2121.27(12)}{79936.06} + 0.5\left(\frac{1588.49}{1120} - \frac{18.67(1588.49)}{12761.88} + \frac{(1714.64+1822.29)(12)}{12761.88} + \frac{(443.72)(12)}{79936.06}\right)$$

$f_t = 0.318 + 0.5(1.418 - 2.323 + 3.326 + 0.067) = 1.563$ ksi

Allowable compression: +2.493 ksi > +1.563 ksi (reqd.) (O.K.)

Concrete stresses at bottom fiber of the beam:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b} - \frac{M_g + M_S}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

$$f_b = \frac{1588.49}{1120} - \frac{18.67(1588.49)}{18024.15} - \frac{(1714.64+1822.29)(12)}{18024.15} - \frac{(443.72+2121.27)(12)}{27842.9}$$

$f_b = 1.418 + 1.645 - 2.36 - 1.098 = -0.397$ ksi

Allowable Tension: 473.7 ksi > 397 psi (O.K.)

Stresses at the top of the slab

Case (I):

$$f_t = \frac{M_{SDL} + M_{LL+I}}{S_{tc}} = \frac{(443.72+2121.27)(12)}{50802.19} = +0.604 \text{ ksi}$$

Allowable compression: +2.4 ksi > +0.604 ksi (reqd.) (O.K.)

Case (II):

$$f_t = \frac{M_{SDL}}{S_{tc}} = \frac{(443.72)(12)}{50802.19} = 0.103 \text{ ksi}$$

Allowable compression: +1.6 ksi > +0.103 ksi (reqd.) (O.K.)

Case (III):

$$f_t = \frac{M_{LL+I} + 0.5(M_{SDL})}{S_{tc}} = \frac{(2121.27)(12) + 0.5(443.72)(12)}{50802.19} = 0.553 \text{ ksi}$$

Allowable compression: +1.6 ksi > +0.553 ksi (reqd.) (O.K.)

B.1.8.2.3 Summary of Stresses at Service Loads

		Top of Slab f_t (ksi)	Top of Beam f_t (ksi)	Bottom of Beam f_b (ksi)
At Midspan	CASE I	+ 0.604	+2.805	
	CASE II	+ 0.103	+2.490	-0.397
	CASE III	+0.553	+1.563	

B.1.8.3 Actual Modular Ratio and Transformed Section Properties for Strength Limit State and Deflection Calculations

Till this point, a modular ratio equal to 1 has been used for the Service Limit State design. For the evaluation of Strength Limit State and Deflection calculations, actual modular ratio will be calculated and the transformed section properties will be used.

$$n = \left(\frac{E_c \text{ for slab}}{E_c \text{ for beam}}\right) = \left(\frac{3834.25}{4531.48}\right) = 0.883$$

Transformed flange width = n (effective flange width) = 0.883(138 in.)
= 121.85 in.

Transformed Flange Area = n (effective flange width) (t_s) = 1(121.85 in.)(8 in.)
= 974.8 in.^{2}

Table B.1.12 Properties of Composite Section

	Transformed Area in.^{2}	y_b in.	$A\ y_b$ in.	$A(y_{bc} - y_b)^2$	I in.^{4}	I+$A(y_{bc}$- $y_b)^2$ in.^{4}
Beam	1,120	22.36	25,043.20	307,883.97	403,020	710,903.97
Slab	974.8	58	56,538.40	354,128.85	41,591	395,720.32
$\sum$	2,094.8		81,581.60			1,106,624.29

A_c = total area of composite section = 2,094.8 in.^{2}

h_c = total height of composite section = 62 in.

I_c = moment of inertia of composite section = 1,106,624.29 in.^{4}

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam = 81,581.6 / 2,094.8 = 38.94 in.

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam = 54 – 38.94 = 15.06 in.

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab = 62 – 38.94 = 23.06 in.

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam
= I_c/y_{bc} = 1,106,624.29 / 38.94 = 28,418.7 in.^{3}

S_{tg} = composite section modulus for top fiber of the precast beam
= I_c/y_{tg} = 1,106,624.29 / 15.06 = 73,418.03 in.^{3}

S_{tc} = composite section modulus for top fiber of the slab
= I_c/y_{tc} = 1,106,624.29 / 23.06 = 47,988.91 in.^{3}

B.1.9

FLEXURAL STRENGTH

[STD Art. 9.17]

Group I load factor design loading combination

$M_u = 1.3[M_g + M_s + M_{SDL} + 1.67(M_{LL+I})]$ [STD Table 3.22.1A]

$= 1.3[1714.64 + 1822.29 + 443.72 + 1.67(2121.27)] = 9780.12$ k-ft.

Average stress in pretensioning steel at ultimate load

$$f^*_{su} = f'_s\left(1 - \frac{\gamma^*}{\beta_1}\rho^*\frac{f'_s}{f'_c}\right)$$ [STD Eq. 9-17]

where,

f^*_{su} = average stress in prestressing steel at ultimate load

γ^* = 0.28 for low-relaxation strand [STD Art. 9.1.2]

$$\beta_1 = 0.85 - 0.05\frac{(f'_c - 4000)}{1000}$$ [STD Art. 8.16.2.7]

$$= 0.85 - 0.05\frac{(4000 - 4000)}{1000} = 0.8$$

$$\rho^* = \frac{A^*_s}{bd}$$

where,

A^*_s = area of pretensioned reinforcement = 66(0.153) = 10.1 in.^{2}

b = transformed effective flange width = 121.85 in.

y_{bs} = distance from center of gravity of the strands to the bottom fiber of the beam = 22.36 – 18.67 = 3.69 in.

d = distance from top of slab to centroid of pretensioning strands

= beam depth (h) + slab thickness – y_{bs}

= 54 + 8 – 3.69 = 58.31 in.

$$\rho^* = \frac{10.1}{121.85(58.31)} = 0.00142$$

$$f^*_{su} = 270\left[1 - \left(\frac{0.28}{0.85}\right)(0.00142)\left(\frac{270}{4}\right)\right] = 261.48 \text{ ksi}$$

Depth of compression block

$$a = \frac{A^*_s f^*_{su}}{0.85 f'_c\, b} = \frac{10.1(261.48)}{0.85(4)(121.85)} = 6.375 \text{ in.} < 8.0 \text{ in.}$$ [STD Art. 9.17.2]

The depth of compression block is less than flange thickness hence the section is designed as rectangular section

Design flexural strength:

$$\phi Mn = \phi A_s^* f_{su}^* d\left(1 - 0.6\frac{\rho^* f_{su}^*}{f_c'}\right)$$ [STD Eq. 9-13]

where,

ϕ = strength reduction factor = 1.0 [STD Art. 9.14]

Mn = nominal moment strength of a section

$$\phi Mn = 1.0(10.1)(261.48)\frac{(58.31)}{12}\left(1\text{-}0.6\ \frac{0.00142(261.48)}{4}\right)$$

= 12118.1 k-ft. > 9780.12 k-ft. (O.K.)

B.1.10 DUCTILITY LIMITS

B.1.10.1 Maximum Reinforcement

[STD Art. 9.18.1]

Reinforcement index for rectangular section:

$$\frac{\rho^* f_{su}^*}{f_c'} < 0.36\,\beta_1 = 0.00142\left(\frac{261.48}{4}\right) = 0.093 < 0.36(0.85) = 0.306$$ (O.K.)

[STD Eq. 9-20]

B.1.10.2 Minimum Reinforcement

[STD Art. 9.18.2]

The ultimate moment at the critical section developed by the pretensioned and non-pretensioned reinforcement shall be at least 1.2 times the cracking moment, M_{cr}

$\phi Mn \geq 1.2\ M_{cr}$

Cracking moment $M_{cr} = (f_r + f_{pe})\ S_{be} - M_{d\text{-}nc}\left(\frac{S_{bc}}{S_b} - 1\right)$ [STD Art. 9.18.2.1]

where,

f_r = modulus of rupture

$$= 7.5\sqrt{f_c'} = 7.5\sqrt{6225}\left(\frac{1}{1000}\right) = 0.592 \text{ ksi}$$ [STD Art. 9.15.2.3]

f_{pe} = compressive stress in concrete due to effective prestress forces at extreme fiber of section where tensile stress is caused by externally applied loads

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b}$$

where,

P_{se} = effective prestress force after losses = 1,583.791 kips

e_c = 18.67 in.

$$f_{pe} = \frac{1588.49}{1120} + \frac{1588.49\ (18.67)}{18024.15} = 1.418 + 1.641 = 3.055 \text{ ksi}$$

$M_{d\text{-}nc}$ = non-composite dead load moment at midspan due to self weight of beam and weight of slab = 1714.64 + 1822.29 = 3536.93 k-ft.

$$M_{cr} = (0.592 + 3.055)(28418.7)\left(\frac{1}{12}\right) - 3536.93\left(\frac{28418.7}{18024.15} - 1\right)$$

$$= 8636.92 - 2039.75 = 6{,}597.165 \text{ k-ft.}$$

$1.2\,M_{cr} = 1.2(6597.165) = 7{,}916.6$ k-ft. $< \phi Mn = 12{,}118.1$ k-ft. (O.K.)

B.1.11 TRANSVERSE SHEAR DESIGN

[STD Art. 9.20]

Members subject to shear shall be designed so that

$V_u < \phi\,(V_c + V_s)$ [STD Eq. 9-26]

where,

V_u = the factored shear force at the section considered

V_c = the nominal shear strength provided by concrete

V_s = the nominal shear strength provided by web reinforcement

ϕ = strength reduction factor = 0.90 [STD Art. 9.14]

The critical section for shear is located at a distance h/2 from the face of the support, however the critical section for shear is conservatively calculated from the center line of the support

$h/2 = \dfrac{62}{2(12)} = 2.583$ ft. [STD Art. 9.20.1.4]

From Tables B.1.3 and Table B.1.4 the shear forces at critical section are as follows,

V_d = Shear force due to total dead loads at section considered = 144.75 kips

V_{LL+I} = Shear force due to live load and impact at critical section = 81.34 kips

$V_u = 1.3(V_d + 1.67V_{LL+I}) = 1.3(144.75 + 1.67(81.34) = 364.764$ kips

Computation of V_{ci}

$$V_{ci} = 0.6\sqrt{f'_c}b'd + V_d + \frac{V_i M_{cr}}{M_{\max}}$$ [STD Eq. 9-27]

where ,

b' = width of web of a flanged member = 5 in.

f'_c = compressive strength of beam concrete at 28 days = 6225 psi.

M_d = bending moment at section due to unfactored dead load = 365.18 k-ft.

M_{LL+I} = factored bending moment at section due to live load and impact = 210.1 k-ft.

M_u = factored bending moment at the section.

$= 1.3(M_d + 1.67M_{LL+I}) = 1.3[365.18 + 1.67(210.1)] = 930.861$ k-ft.

V_{mu} = factored shear force occurring simultaneously with M_u conservatively taken as maximum shear load at the section = 364.764 kips.

M_{max} = maximum factored moment at the section due to externally applied loads = $M_u - M_d = 930.861 - 365.18 = 565.681$ k-ft.

V_i = factored shear force at the section due to externally applied loads occurring simultaneously with M_{max}

$= V_{mu}\text{-}V_d = 364.764 - 144.75 = 220.014$ kips

f_{pe} = compressive stress in concrete due to effective pretension forces at extreme fiber of section where tensile stress is caused by externally applied loads i.e. bottom of the beam in present case

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}\, e}{S_b}$$

eccentricity of the strands at $h_c/2$

$e_{h/2} = 18.046$ in.

P_{se} = 36(0.153)(157.307) = 866.45 kips

$$f_{pe} = \frac{866.45}{1120} + \frac{866.45(17.95)}{18024.15} = 0.77 + 0.86 = 1.63 \text{ ksi}$$

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \left[\frac{M_g + M_S}{S_b} + \frac{M_{SDL}}{S_{bc}}\right]$$

$$= \left[\frac{(159.51 + 157.19+7.75)(12)}{18024.15} + \frac{41.28(12)}{28418.70}\right] = 0.234 \text{ ksi}$$

M_{cr} = moment causing flexural cracking of section due to externally applied loads = $(6\, f_c' + f_{pe} - f_d)\, S_{bc}$ [STD Eq. 9-28]

$$= \left(\frac{6\sqrt{6225}}{1000} + 1.631 - 0.234\right)\frac{28418.70}{12} = 4429.5 \text{ k-ft.}$$

d = distance from extreme compressive fiber to centroid of Pretensioned reinforcement, but not less than $0.8h_c$ = 49.6 in.

= 62 – 4.41 = 57.59 in. > 49.96 in.

Therefore, use = 57.59 in.

$$V_{ci} = 0.6\sqrt{f_c'}\, b'd + V_d + \frac{V_i M_{cr}}{M_{\max}}$$ [STD Eq. 9-27]

$$= \frac{0.6\sqrt{6225}(2\times5)(57.59)}{1000} + 144.75 + \frac{220.014(4429.5)}{565.681} = 1894.81 \text{ kips}$$

This value should not be less than

Minimum $V_{ci} = 1.7\sqrt{f_c'}\, b'd$ [STD Art. 9.20.2.2]

$$= \frac{1.7\sqrt{6225}(2\times5)(57.59)}{1000} = 77.24 \text{ kips} < V_{ci} = 1894.81 \text{ kips} \quad \text{(O.K.)}$$

Computation of V_{cw} [STD Art. 9.20.2.3]

$V_{cw} = (3.5\sqrt{f_c'} + 0.3 f_{pc})b'd + V_p$ [STD Eq. 9-29]

where,

f_{pc} = compressive stress in concrete at centroid of cross-section (Since the centroid of the composite section does not lie within the flange of the cross-section) resisting externally applied loads. For a non-composite section

$$f_{pc} = \frac{P_{se}}{A} - \frac{P_{se}\, e(y_{bc} - y_b)}{I} + \frac{M_D(y_{bc} - y_b)}{I}$$

M_D = moment due to unfactored non-composite dead loads = 324.45 k-ft.

$$f_{pc} = \frac{863.89}{1120} - \frac{863.89\,(17.95)(38.94\text{-}22.36)}{403020} + \frac{324.45(12)(38.94\text{-}22.36)}{403020}$$

$$= 0.771 - 0.638 + 0.160 = 0.293 \text{ psi}$$

$$V_p = 0$$

$$V_{cw} = \left(\frac{3.5\sqrt{6225}}{1000} + 0.3(0.293)\right)(2\times 5)(57.59) = 209.65 \text{ kips (controls)}$$

The allowable nominal shear strength provided by concrete should be lesser of V_{ci} = 1894.81 kips and V_{cw} = 209.65 kips

Therefore, V_c = 209.65 kips

$V_u < \phi\,(V_c + V_s)$

where, ϕ = strength reduction factor for shear = 0.90

$$\text{Required } V_s = \frac{V_u}{\phi} - V_c = \frac{364.764}{0.9} - 209.65 = 195.643 \text{ kips}$$

Maximum shear force that can be carried by reinforcement

$V_{s\,max} = 8\sqrt{f_c'}\,b'd$ [STD Art. 9.20.3.1]

$$= 8\sqrt{6225}\,\frac{(2\times 5)(57.59)}{1000} = 363.502 \text{ kips} > \text{required } V_s = 195.643 \text{ kips (O.K.)}$$

Area of shear steel required [STD Art. 9.20.3.1]

$$V_s = \frac{A_v f_y d}{s}$$ [STD Eq. 9-30]

$$\text{or } A_v = \frac{V_s\, s}{f_y\, d}$$

where,

A_v = area of web reinforcement, in.^{2}

s = longitudinal spacing of the web reinforcement, in.

Setting s = 12 in. to have units of in.^{2}/ft. for A_v

$$A_v = \frac{(195.643)(12)}{(60)(57.59)} = 0.6794 \text{ in.}^2/\text{ft.}$$

Minimum shear reinforcement [STD Art. 9.20.3.3]

$$A_{v-min} = \frac{50\,b'\,s}{f_y} = \frac{(50)(2\times 5)(12)}{60000} = 0.1 \text{ in.}^2/\text{ft.}$$ [STD Eq. 9-31]

The required shear reinforcement is the maximum of A_v = 0.378 in.^{2}/ft. and

$A_{v-min} = 0.054$ in.^{2}/ft.

[STD Art. 9.20.3.2]

Maximum spacing of web reinforcement is 0.75 h_c or 24 in., unless

$$V_s = 195.643 \text{ kips} > 4\sqrt{f_c'}\, b'\, d = 4\sqrt{6225}\ \frac{(2\times5)(57.59)}{1000} = 181.751 \text{ kips}$$

Use 1 # 4 double legged with $A_v = 0.392$ in.^{2} / ft., the required spacing can be calculated as,

$$s = \frac{f_y\, d\, A_v}{V_s} = \frac{60\times57.59\times0.392}{195.643} = 6.92 \text{ in.}$$

Since, V_s is less than the limit,

Maximum spacing = 0.75 h = 0.75(54 + 8 + 1.5) = 47.63 in.

or = 24 in.

Therefore, maximum s = 24 in.

Use # 4, two legged stirrups at 6.5 in. spacing.

B.1.12 HORIZONTAL SHEAR DESIGN

[STD Art. 9.20.4]

The critical section for horizontal shear is at a distance of $h_c/2$ from the center line of the support

$V_u = 364.764$ kips

$V_u \leq V_{nh}$ [STD Eq. 9-31a]

where, V_{nh} = nominal horizontal shear strength, kips

$$V_{nh} \geq \frac{V_u}{\phi} = \frac{364.764}{0.9} = 405.293 \text{ kips}$$

Case (a & b): Contact surface is roughened, or when minimum ties are used

Allowable shear force: [STD Art. 9.20.4.3]

$V_{nh} = 80 b_v d$

where,

b_v = width of cross-section at the contact surface being investigated . for horizontal shear = 2×15.75= 31.5 in.

d = distance from extreme compressive fiber to centroid of the pretensioning force = 54 –4.41 = 49.59 in.

$$V_{nh} = \frac{80(31.5)(49.59)}{1000} = 124.97 \text{ kips} < 405.293 \text{ kips} \qquad \text{(N.G.)}$$

Case(c): Minimum ties provided, and contact surface roughened

Allowable shear force: [STD Art. 9.20.4.3]

$$V_{nh} = 350 b_v d$$

$$= \frac{350(31.5)(49.59)}{1000} = 546.73 \text{ kips} > 405.293 \text{ kips} \qquad \text{(O.K.)}$$

Required number of stirrups for horizontal shear [STD Art. 9.20.4.5]

$$\text{Minimum } A_{vh} = 50\frac{b_v s}{f_y} = 50\frac{(31.5)(6.5)}{60000} = 0.171 \text{ in.}^2\text{/ft.}$$

Therefore, extend every alternate web reinforcement into the cast-in-place slab to satisfy the horizontal shear requirements.

Maximum spacing = $4b = 4(2 \times 15.75) = 126$ in. [STD Art. 9.20.4.5.a]

or = 24.00 in.

Maximum spacing = 24 in. > ($s_{provided}$ = 13.00 in.)

B.1.13 PRETENSIONED ANCHORAGE ZONE

[STD Art. 9.22]

B.1.13.1 Minimum Vertical Reinforcement

In a pretensioned beam, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total pretensioning force must be placed within the distance of $d/4$ of the beam end. [STD Art. 9.22.1]

Minimum stirrups at the each end of the beam:

P_s = prestress force before initial losses = 36(0.153)[(0.75)(270)] = 1,115.37 kips

4% of P_s = 0.04(1115.37) = 44.62 kips

$$\text{Required } A_v = \frac{44.62}{20} = 2.231 \text{ in.}^2$$

$$\frac{d}{4} = \frac{57.59}{4} = 14.4 \text{ in.}$$

Use 5 pairs of #5 @ 2.5 in. spacing at each end of the beam (provided A_v = 3.1 in.^{2})

Provide nominal reinforcement to enclose the pretensioning steel for a distance from the end of the beam equal to the depth of the beam [STD Art. 9.22.2]

B.1.14 DEFLECTION AND CAMBER

B.1.14.1 Maximum Camber Calculations Using Hyperbolic Functions Method

TxDOT's prestressed bridge design software, PSTRS 14 uses the Hyperbolic Functions Method proposed by Sinno Rauf, and Howard L Furr (1970) for the calculation of maximum camber. This design example illustrates the PSTRS 14 methodology for calculation of maximum camber.

Step1: Total prestress after release

$$P = \frac{P_{si}}{\left(1 + pn + \dfrac{e_c^2 A_s n}{I}\right)} + \frac{M_D e_c A_s n}{I\left(1 + pn + \dfrac{e_c^2 A_s n}{I}\right)}$$

where,

P_{si} = total prestressing force = 1,881.146 kips

I = moment of inertia of non-composite section = 403,020 in.^{4}

e_c = eccentricity of pretensioning force at the midspan = 18.67 in.

M_D = Moment due to self weight of the beam at midspan = 1,714.64 k-ft.

A_s = Area of strands = number of strands (area of each strand)

= 66(0.153) = 10.098 in.^{2}

$p = A_s/A$

where,

A = Area of cross-section of beam = 1,120 in.^{2}

p = 10.098/1120 = 0.009016

E_c = modulus of elasticity of the beam concrete at release, ksi

$= 33(w_c)^{3/2}\sqrt{f_c'}$ [STD Eq. 9-8]

$= 33(150)^{1.5}\sqrt{5140}\ \dfrac{1}{1000} = 4{,}346.43$ ksi

E_s = Modulus of elasticity of prestressing strands = 28000 ksi

$n = E_s/E_c$ = 28000/4346.43 = 6.45

$$\left(1 + pn + \frac{e_c^2 A_s n}{I}\right) = 1 + (0.009016)(6.45) + \frac{(18.67^2)(10.098)(6.45)}{403020} = 1.115$$

$$P = \frac{P_{si}}{\left(1 + pn + \dfrac{e_c^2 A_s n}{I}\right)} + \frac{M_D e_c A_s n}{I\left(1 + pn + \dfrac{e_c^2 A_s n}{I}\right)}$$

$$= \frac{1881.15}{1.115} + \frac{(1714.64)(12\text{ in./ft.})(18.67)(10.098)(6.45)}{403020(1.115)}$$

= 1687.13 + 55.68 = 1,742.81 kips

Concrete stress at steel level immediately after transfer

$$f_{ci}^{s} = P\left(\frac{1}{A} + \frac{ec^2}{I}\right) - f_c^s$$

where,

f_c^s = Concrete stress at steel level due to dead loads

$$= \frac{M_D\, e_c}{I} = \frac{(1714.64)(12\text{ in./ft.})(18.67)}{403020} = 0.953 \text{ ksi}$$

$$f_{ci}^{s} = 1742.81\left(\frac{1}{1120} + \frac{18.67^2}{403020}\right) - 0.953 = 2.105 \text{ ksi}$$

Step2: Ultimate time-dependent strain at steel level

$$\varepsilon_{c1}^{s} = \varepsilon_{cr}^{\infty} f_{ci}^{s} + \varepsilon_{sh}^{\infty}$$

where,

$\varepsilon_{cr}^{\infty}$ = ultimate unit creep strain = 0.00034 in./in. (this value is prescribed by Sinno et. al. (1970)

$\varepsilon_{sh}^{\infty}$ = ultimate unit creep strain = 0.000175 in./in. (this value is prescribed by Sinno et. al. (1970))

$\varepsilon_{c1}^{\infty}$ = 0.00034(2.105) + 0.000175 = 0.0008907 in./in.

Step3: Adjustment of total strain in step 2

$$\varepsilon_{c2}^{s} = \varepsilon_{c1}^{s} - \varepsilon_{c1}^{s} E_{ps}\frac{A_s}{E_{ci}}\left(\frac{1}{A_n} + \frac{ec^2}{I}\right)$$

$$= 0.0008907 - 0.0008907\,(28000)\frac{10.098}{4346.43}\left(\frac{1}{1120} + \frac{18.67^2}{403020}\right) = 0.000993 \text{ in./in.}$$

Step4: Change in concrete stress at steel level

$$\Delta f_c^s = \varepsilon_{c2}^{s} E_{ps} A_s\left(\frac{1}{A_n} + \frac{ec^2}{I}\right) = 0.000993\,(28000)(10.098)\left(\frac{1}{1120} + \frac{18.67^2}{403020}\right)$$

Δf_c^s = 0.494 ksi

Step5: Correction of the total strain from step2

$$\varepsilon^{s}_{c4} = \varepsilon^{\infty}_{cr} + \left(f^{s}_{ci} - \frac{\Delta f^{s}_{c}}{2} \right) + \varepsilon^{\infty}_{sh}$$

$$\varepsilon^{s}_{c4} = 0.00034\left(2.105 - \frac{0.494}{2} \right) + 0.000175 = 0.000807 \text{ in./in.}$$

Step6: Adjustment in total strain from step 5

$$\varepsilon^{s}_{c5} = \varepsilon^{s}_{c4} - \varepsilon^{s}_{c4} E_{ps} \frac{A_s}{E_c}\left(\frac{1}{A_n} + \frac{e_c^{2}}{I} \right)$$

$$= 0.000807 - 0.000807(28000)\frac{10.098}{4346.43}\left(\frac{1}{1120} + \frac{18.67^{2}}{403020} \right) = 0.000715 \text{ in./in.}$$

Step 7: Change in concrete stress at steel level

$$\Delta f^{s}_{c1} = \varepsilon^{s}_{c5} E_{ps} A_s \left(\frac{1}{A_n} + \frac{e_c^{2}}{I} \right) = 0.000715\ (28000)(10.098) \left(\frac{1}{1120} + \frac{18.67^{2}}{403020} \right)$$

$$\Delta f^{s}_{c1} = 0.36 \text{ ksi}$$

Step 8: Correction of the total strain from step 5

$$\varepsilon^{s}_{c6} = \varepsilon^{\infty}_{cr} + \left(f^{s}_{ci} - \frac{\Delta f^{s}_{c1}}{2} \right) + \varepsilon^{\infty}_{sh}$$

$$\varepsilon^{s}_{c6} = 0.00034\left(2.105 - \frac{0.36}{2} \right) + 0.000175 = 0.00083 \text{ in./in.}$$

Step9: Adjustment in total strain from step 8

$$\varepsilon^{s}_{c7} = \varepsilon^{s}_{c6} - \varepsilon^{s}_{c6} E_{ps} \frac{A_s}{E_{ci}}\left(\frac{1}{A_n} + \frac{e_c^{2}}{I} \right)$$

$$= 0.00083 - 0.00083\ (28000)\ \frac{10.098}{4346.43}\left(\frac{1}{1120} + \frac{18.67^{2}}{403020} \right) = 0.000735 \text{ in./in.}$$

Step 10: Computation of initial prestress loss

$$PL_i = \frac{P_{si} - P}{P_{si}} = \frac{1877.68 - 1742.81}{1877.68} = 0.0735$$

Step 11: Computation of Final Prestress loss

$$PL^{\infty} = \frac{\varepsilon_{c7}^{\infty}\ E_{ps}\ A_s}{P_{si}} = \frac{0.000735(28000)(10.098)}{1877.68} = 0.111$$

Total Prestress loss

$$PL = PL_i + PL^{\infty} = 100(0.0735 + 0.111) = 18.45\%$$

Step 12: Initial deflection due to dead load

$$C_{DL} = \frac{5\ w\ L^4}{384\ E_c\ I}$$

where,

w = weight of beam = 1.167 kips/ft.

L = span length = 108.417 ft.

$$C_{DL} = \frac{5\left(\frac{1.167}{12 \text{ in./ft.}}\right)[(108.417)(12 \text{ in./ft.})]^4}{384(4346.43)(403020)} = 2.073 \text{ in.}$$

Step 13: Initial Camber due to prestress

M/EI diagram is drawn for the moment caused by the initial prestressing, is shown in Figure B.1.9. Due to debonding of strands, the number of strands vary at each debonding section location. Strands that are bonded, achieve their effective prestress level at the end of transfer length. Points 1 through 6 show the end of transfer length for the preceding section. The *M/EI* values are calculated as,

$$\frac{M}{EI} = \frac{P_{si} \times ec}{E_c I}$$

The *M/EI* values are calculated for each point 1 through 6 and are shown in Table B.1.14. The initial camber due to prestress, C_{pi}, can be calculated by Moment Area Method, by taking the moment of the *M/EI* diagram about the end of the beam.

C_{pi} = 4.06 in.

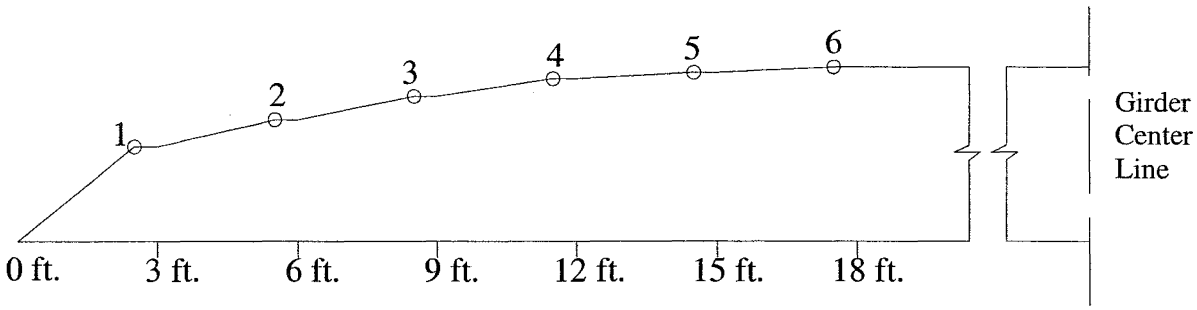

***Figure B.1.9** M/EI Diagram to Calculate the Initial Camber due to Prestress*

Table B.1.13 *M/EI* Values at the End of Transfer Length

Identifier for the End of Transfer Length	P_{si} (kips)	ec (in.)	M/EI (in.^{3})
1	1024.19	17.95	1.026E-08
2	1308.69	18.01	1.029E-08
3	1536.29	18.33	1.048E-08
4	1763.88	18.57	1.061E-08
5	1820.78	18.62	1.064E-08
6	1877.68	18.67	1.067E-08

Step 14: Initial Camber

$C_i = C_{pi} - C_{DL} = 4.06 - 2.073 = 1.987$ in.

Step 15: Ultimate Time Dependent Camber

Ultimate strain $\varepsilon_e^s = \dfrac{f_{ci}^s}{E_c}$ = 2.105/4346.43 = 0.00049 in./in.

Ultimate camber $C_t = C_i\left(1 - PL^{\infty}\right)\dfrac{\varepsilon_{cr}^{\infty}\left(f_{ci}^s - \dfrac{\Delta f_{c1}^s}{2}\right) + \varepsilon_e^s}{\varepsilon_e^s}$

$$= 1.987(1 - 0.111)\frac{0.00034\left(2.105 - \dfrac{0.494}{2}\right) + 0.00049}{0.00049}$$

$C_t = 4.044$ in. = 0.34 ft. ↑

B.1.14.2
Deflection due to Beam Self-Weight

$$\Delta_{beam} = \frac{5w_g L^4}{384 E_{ci} I}$$

where, w_g = beam weight = 1.167 kips/ft.

Deflection due to beam self weight at transfer

$$\Delta_{beam} = \frac{5(1.167/12)[(109.5)(12)]^4}{384(4346.43)(403020)} = 2.16 \text{ in.} \downarrow$$

Deflection due to beam self-weight used to compute deflection at erection

$$\Delta_{beam} = \frac{5(1.167/12)[(108.4167)(12)]^4}{384(4783.22)(403020)} = 1.88 \text{ in.} \downarrow$$

B.1.14.3
Deflection due to Slab and Diaphragm Weight

$$\Delta_{slab} = \frac{5 w_s L^4}{384 E_c I} + \frac{w_{dia} b}{24 E_c I}\left(3l^2 - 4b^2\right)$$

where,

w_s = slab weight = 1.15 kips/ft.

E_c = modulus of elasticity of beam concrete at service = 4,783.22 ksi

$$\Delta_{slab} = \frac{5(1.15/12)[(108.4167)(12)]^4}{384(4783.22)(403020)} + \frac{(3)(44.2083 \times 12)}{(24 \times 4783.22 \times 403020)}\left(3(108.4167 \times 12)^2 - 4(44.2083 \times 12)^2\right)$$

$= 1.99$ in. $\downarrow$

B.1.14.4
Deflection due to Superimposed Loads

$$\Delta_{SDL} = \frac{5 w_{SDL} L^4}{384 E_c I_c}$$

where,

w_{SDL} = super imposed dead load = 0.31 kips/ft.

I_c = moment of inertia of composite section = 1,106,624.29 in.^{4}

$$\Delta_{SDL} = \frac{5(0.302/12)[(108.4167)(12)]^4}{384(4783.22)(1106624.29)} = 0.18 \text{ in.} \downarrow$$

Total deflection at service due to all dead loads = 1.88 + 1.99 + 0.18
= 4.05 in. = 0.34 ft.

B.1.14.5
Deflection due to Live Load and Impact

The deflections due to live loads are not calculated in this example as they are not a design factor for TxDOT bridges.

B.1.15 COMPARISON OF RESULTS

In order to measure the level of accuracy in this detailed design example, the results are compared with that of PSTRS14 (TxDOT 2004). The summary of comparison is shown in Table B.1.15. In the service limit state design, the results of this example matches those of PSTRS14 with very insignificant differences. A difference of 26 percent in transverse shear stirrup spacing is observed. This difference can be because of the fact that PSTRS14 calculates the spacing according to the AASHTO Standard Specifications 1989 edition (AASHTO 1989) and in this detailed design example, all the calculations were performed according to the AASHTO Standard Specifications 2002 edition (AASHTO 2002). There is a difference of 15.3 percent in camber calculation, which can be due to the fact that PSTRS14 uses a single step hyperbolic functions method, whereas, a multi step approach is used in this detailed design example.

Table B.1.14 Comparison of Results for the AASHTO Standard Specifications (PSTRS14 vs Detailed Design Example)

Design Parameters		PSTRS14	Detailed Design Example	% diff. w.rt. PSTRS14
Prestress Losses, (%)	Initial	8.00	8.01	-0.1
	Final	22.32	22.32	0.0
Required Concrete Strengths, (psi)	f'_{ci}	5,140	5,140	0.0
	f'_c	6,223	6,225	0.0
At Transfer (ends), (psi)	Top	-530	-526	0.8
	Bottom	1,938	1,935	0.2
At Service (midspan), (psi)	Top	-402	-397	1.2
	Bottom	2,810	2,805	0.2
Number of Strands		66	66	0.0
Number of Debonded Strands		(20+10)	(20+10)	0.0
M_u, (kip–ft.)		9,801	9,780.12	0.3
ϕM_n, (kip–ft.)		12,086	12,118.1	-0.3
Transverse Shear Stirrup (#4 bar) Spacing, (in.)		8.8	6.5	26.1
Maximum Camber, (ft.)		0.295	0.34	-15.3

Example of Tx DOT Standard AASHTO IV Bridge 44' RDWY X 110 Span

File Name: LRFDexplE20.mcd this is a by the book.

LRFD Cap Design Example for Bridge standard Type IV I-Beam 110' Span, 44' roadway

Input [Answers]

Span Properties

$RoadwayWidth := 44 \ ft$

$OverAllwidth := 46 \ ft$

$Span := 110 \ ft$

$BeamSpace := 8 \ ft$

$NumberOfBeams := 6$

$BeamLength := 109.67 \ ft$

$Skew := 0$

Take the Longer of the spans to calculate loads for the bent design. This ignores any possible torsion from vertical loads.

Constants

NG := "NG"

OK := "OK"

Cap Dimensions

$CapWidth := 3.25 \ ft$

$CapDepth := 3.25 \ ft$

$CapLength := 44 \ ft$

Column Dimensions

$ColumnDiameter := 3.0 \ ft$

$ColumnSpace := 17.0 \ ft$

$NumberOfColumns := 3$

$ColumnHeight := 20 \ ft$

Dead Load Constants

$RailWeight := .326 \ klf$

Rail Weight is based on Tx Dot T-501

$BeamWeight := .821 \ klf$

Beam Weight is based on AASHTO Ty IV

$Overlay := 2 \ in$

Overlay, 2" for the example is an accepted value

Reinforced Concrete Properties

$fc := 3.6 \ ksi$

Class C concrete 3,600 psi

$fy := 60 \ ksi$

$Ec := 33000 \cdot \left[\left(0.145^{1.5} \right) \cdot \sqrt{fc} \right]$ $\quad Ec = 3.457 \times 10^{3} \ ksi$

For Normal Weight Concrete use Ec1 LRFD 5.4.2.4-1

$Es := 29000 \ ksi$

Es LRFD 5.4.3.2

Ec=1820* (f'c)^2 is the simplified form

$Ec1 := 1820 \cdot \sqrt{fc}$

$Ec1 = 3.453 \times 10^{3} \ ksi$

Design Lanes LRFD 3.6.1.1.1

$$NoOfLanes := \frac{RoadwayWidth}{12}$$

$trunc(NoOfLanes) = 3$

Multiple presence Factor

$Mpf1 := 1.2$
$Mpf2 := 1.0$
$Mpf3 := 0.85$

$MaxLanes := trunc(NoOfLanes)$

$Mpf4 := 0.65$

$\boxed{MaxLanes = 3}$

Breaking Force LRFD 3.6.4

$BR1 := .25\cdot(32 + 32 + 8)\cdot MaxLanes\cdot Mpf3$

$\boxed{BR1 = 45.9}$ kips

BR1 This represents 25 % of the truck load 3 lanes.

$BR2a := MaxLanes\cdot Mpf3\cdot 0.05\cdot[\,[\,72 + (Span + Span)\cdot 0.64\,]\,]$

$\boxed{BR2a = 27.132}$ kips

BR2 is 5% of the design truck plus lane or 5% of the tandem plus the lane

$BR2b := MaxLanes\cdot Mpf3\cdot 0.05\cdot[(25 + 25) + 2\cdot Span\cdot .64]$

$\boxed{BR2b = 24.327}$ kips

Use the greater of the tandem or truck for design.

Shrinkage LRFD 3.12.4

Due to the symmetry of the bridge superstructure, no force is developed at the intermediate bent due to the shrinkage of the superstructure.

Dead Load

Rail: $DLr := RailWeight\cdot\frac{Span}{2}$

Note: Rail weight has been devided between the two outermost beams.

$\boxed{DLr = 17.93}$ kips/beam pair

Dead Load Rail based on T-501

Slab: $ConcreteWt := .15$ kip/cf

$SlabConcrete := 130.2\,cy$

The cy volumes are from the standards

$$DLs := 27\cdot SlabConcrete\cdot\frac{ConcreteWt\cdot 1.05}{NumberOfBeams}$$

Add 5% for haunch and thickened end Slabs

This is the load for the reaction of both beams

$\boxed{DLs = 92.279}$ kips/beam pair

Beam: $DLb := BeamWeight\cdot BeamLength$

$\boxed{DLb = 90.039}$ kips/beam pair

Overlay: $AsphaltWt := .14$ kip/cf

$$Dwol := \frac{AsphaltWt\cdot Overlay\cdot BeamSpace\cdot Span}{12}$$

$Dwol = 20.533$ kips/beam pair

TxDOT Standard IBA 1998

$DW := Dwol$

$\boxed{DW = 20.533}$ kips/beam pair

Cap:

$Station := 0.5$ ft/sta

Station for the incremental load used in Cap 18 is set at 1/2 foot

$DLcap := CapWidth \cdot CapDepth \cdot ConcreteWt \cdot Station$

$\boxed{DLcap = 0.792}$ Cap 18 input

Dead load total per beam pair:

DC defined as dead loads that are considered composite with the decks and beams or part of the clearly defined permanent loads: Slab +Beam +Rail.

$DC := DLr + DLs + DLb$

$DC = 200.248$ kips/beam pair

This result is a combination of span one and span two

$$DL18F := \frac{(DC \cdot 1.25 + DLcap \cdot 1.25 + DW \cdot 1.5)}{DC + DW + DLcap}$$

$\boxed{DL18F = 1.273}$ Cap 18 factor

DL18F adjusts all deadload factors to one

$DLtotal := DLr + DLs + DLb + Dwol$

$\boxed{DLtotal = 220.782}$ kip/beam Cap 18 input

This is the Cap 18 input for the Dead load of the cap based on .5 ft length of cap

Live Load

$IM := 1.33$

Dynamic Load Allowable (Table LRFD 3.6.2.1-1) applied to the truck load or tandem Load as Specified in LRFD 3.6.1.2.4

Lane:

$LaneLoad := .64 \cdot Span$

$\boxed{LaneLoad = 70.4}$ kip

For developing Standards use the Long Span if the span lengths are different

Truck: $Truck := 32 + 32\left(\frac{Span - 14}{Span}\right) + 8 \cdot \left(\frac{Span - 14}{Span}\right)$

$\boxed{Truck = 66.909}$ kip

$$TruckTrain := \left[32 + 32 \cdot \left(\frac{Span - 14}{Span}\right) + 8 \cdot \left(\frac{Span - 28}{Span}\right) + 8 \cdot \left(\frac{Span - 50}{Span}\right) + 32 \cdot \left(\frac{Span - 64}{Span}\right) + 32 \cdot \left(\frac{Span - 78}{Span}\right)\right]$$

$\boxed{TruckTrain = 92.945}$ kip

$ControlTruck := if(Truck \geq TruckTrain, Truck, TruckTrain)$

$\boxed{ControlTruck = 92.945}$ kip

LRFD uses a combination of Lane and Truck Load. The impact is applied to the Truck only LRFD 3.6.1.3 Truck Train takes the place of the old point load and controls over 80'

$LLRxn := 0.9 \cdot (LaneLoad + ControlTruck \cdot IM)$

$\boxed{LLRxn = 174.616}$ kip

$$P1 := \frac{(ControlTruck \cdot IM) \cdot 0.9}{2}$$

$\boxed{P1 = 55.628}$ kip Cap 18 input

$$w := \frac{(LaneLoad) \cdot 0.90}{20}$$ $\boxed{w = 3.168}$ Cap 18 input

This Step differs from TxDOT interpretation.TX distributes the Truck load to maintain a 32 axle load

20 stations represents 10 feet that the lane load is distributed over

Cap 18 Input Data

Multiple presence Factors, m LRFD 3.6.1.1.2
Number of Lanes 1Ln=1.2, 2Ln=1.0, 3Ln=0.85, greater than 3Ln =0.65

Note when using CAP 18 for LRFD an additional analysis will need to be performed that defines one large lane as the clear width of the bridge. This corrects the 1.2 multiple presence factor for a single lane in the random load calculation feature of the program.

Limit States LRFD 3.4.1

For cap design consider Strength 1 and Service 1

DC dead load of permanent components

Dw is wearing surface components

LL is lane load plus the Truck load*1.33 impact

BR breaking force transferred from superstructure

$Mr = \phi\ Mn$ LRFD 5.5.4.2-1

bw is b

Cap 18 only takes one factor for dead load so the DC and DW are combined into one load with a modified load factor DL18F

Strength 1 DC*1.25+DW*1.5+LL*1.75

=DL18F(DC+DW)+(P1+W)*1.75

Service 1 $DC \cdot 1.0 + DW \cdot 1.0 + LL \cdot 1.0$

Tx DOT checks service level dead load to minimize the development of cracks.

Cap 18 Output (moments)

	(kip − ft) Max + M	Sta	(kip − ft) Max - M	Sta
Dead load	posDL := 365.0	70	negDL := 682.2	80
Service	posServ := 799.5	70	negServ := 1063.9	80
Ultimate	posUlt := 1181.7	70	negUlt := 1520.8	80

Moment Summary from Cap 18

Max Moments

$Mupos := posUlt$ $\boxed{Mupos = 1.182 \times 10^{3}}$ kip-ft

$Muneg := negUlt$ $\boxed{Muneg = 1.521 \times 10^3}$ kip-ft

Minimum Flexural Reinforcement LRFD 5.7.3.3.2

For minimum reinforcement Mr must be equal to the lesser of the two equations Mcr or 1.33 Mu

$Ig := (CapWidth \cdot 12) \cdot \frac{(CapDepth \cdot 12)^3}{12}$

$Ig = 1.928 \times 10^5$ in^4

Mr factored resistance
Mn nominal resistance 5.7.3.2.2
ϕ Resistance factor 5.5.4.2
Mu ultimate moment
Ig Section moment of inertia

$fr := 0.24 \cdot \sqrt{fc}$ $fr = 0.455$ psi

$yt := CapDepth \cdot \frac{12}{2}$ $yt = 19.5$

$Mcr := Ig \cdot \frac{fr}{yt}$ $Mcr = 4.502 \times 10^3$ kip-in

$Mcr1 := 1.2 \cdot \frac{Mcr}{12}$ $Mcr1 = 450.2$ kip ft

$Mcr2 := 1.33 \cdot posUlt$ $Mcr2 = 1.572 \times 10^3$ kip ft

$Mcr3 := 1.33 \cdot negUlt$ $Mcr3 = 2.023 \times 10^3$ kip ft

$Mfpos := if(Mcr1 \le Mcr2, Mcr1, Mcr2)$

$\boxed{Mfpos = 450.2}$ kip ft

$Mfneg := if(Mcr1 \le Mcr3, Mcr1, Mcr3)$

$\boxed{Mfneg = 450.2}$ kip ft

LRFD 5.7.3.6.2

1.2*Mcr is Mcr1

1.33 Mu is Mcr2 or Mcr3

Mf is minim flexural reinforcement

$No11 := 1.56$ in^2

Moment Capacity Design LRFD 5.7.3.2

LRFD 5.7.2.2

B or AsB, the B will stand for bottom steel (positive moments)
bw is taken as b , Mn is determined by using LRFD 5.7.3.1.1-1 through 5.7.3.2.2-1 (rectangular sections)
Compression reinforcement is neglected in the calculation of flexural resistance.

$\phi := 0.9$ $\beta1 := 0.85$

$BarNo := 6$ Top

$BarNoB := 5$ Bottom

$As := BarNo \cdot No11$ $As = 9.36$ in^2

$AsB := BarNoB \cdot No11$ $AsB = 7.8$ in^2

$d := (CapDepth \cdot 12) - 2 - \left(\frac{5}{8}\right) - \frac{1.41}{2}$ $\boxed{d = 35.67}$ in

$b := CapWidth \cdot 12$ $b = 39$ in

$fc = 3.6$ ksi $fy := 60$ ksi

LRFD finds a using c 5.7.3.1.2-4

$c := \frac{As \cdot fy}{.85 \cdot fc \cdot \beta1 \cdot b}$ $c = 5.536$ in

c for upper steel

$cB := \frac{AsB \cdot fy}{.85 \cdot fc \cdot \beta1 \cdot b}$ $cB = 4.614$ in

cB for bottom steel

$a := c \cdot \beta1$ $a = 4.706$ in

$aB := cB \cdot \beta1$ $aB = 3.922$ in

Nominal Resistance

$Mn := As \cdot fy \cdot \left(d - \frac{a}{2}\right)$ $\boxed{Mn = 1.871 \times 10^4}$ kip in LRFD 5.7.3.2.2-1 Top Steel

$MnB := AsB \cdot fy \cdot \left(d - \frac{aB}{2}\right)$ $\boxed{MnB = 1.578 \times 10^4}$ kip in LRFD 5.7.3.2.2-1 Bottom Steel

Flexural Resistance

$Mr := \phi \cdot \frac{Mn}{12}$ $\boxed{Mr = 1.403 \times 10^3}$ kip ft Resistance provided by the section for negative moment Top Steel

$MrB := \phi \cdot \frac{MnB}{12}$ $\boxed{MrB = 1.183 \times 10^3}$ kip ft Resistance provided by the section for positive moment Bottom Steel

Ultimate $posUlt = 1.182 \times 10^3$ kip ft Repeat information

$negUlt = 1.521 \times 10^3$ kip ft

$MinFlexPos := if[(MrB \geq Mupos), OK, NG]$ Positive Reinforcement check

$\boxed{MinFlexPos = \text{"OK"}}$

$MinFlexNeg := if[(Mr \geq Muneg), OK, NG]$ Negative reinforcement check

$\boxed{MinFlexNeg = \text{"NG"}}$

Check As Min Top

$MinReinf := if[(Mr \geq Mfneg), OK, NG]$ Minimum Reinforcement Check LRFD 5.7.3.3.2

$\boxed{MinReinf = \text{"OK"}}$

Check As Min Bottom

$MinReinfB := if[(MrB \geq Mfpos), OK, NG]$ Minimum Reinforcement Check LRFD 5.7.3.3.2

$\boxed{MinReinfB = \text{"OK"}}$

Check As Top Max LRFD 5.7.3.3.1-1 $\frac{c}{d} < 0.42$

$TopcdRatio := \frac{c}{d}$ $TopcdRatio = 0.155$

$TopMaxSteel := if[(TopcdRatio \leq 0.42), OK, NG]$

$\boxed{TopMaxSteel = \text{"OK"}}$ LRFD 5.7.3.3.1-1 $\frac{c}{d} < 0.42$

Check As Bottom Max

$BottomcdRatio := \frac{cB}{d}$ $BottomcdRatio = 0.129$

$BottomMaxSteel := if(BottomcdRatio \le 0.42, OK, NG)$

$BottomMaxSteel = "OK"$

Check Serviceability Top

Control of cracking by distribution reinforcement LRFD 5.7.3.4

Clear Cover is 2 inches or less

dc is distance from extreme tension fiber to center of bar located closest thereto.

ds is the centroid of the tensile reinfocement. For one steel layer dc=ds.

$dc := 2 + \left(\frac{5}{8}\right) + \frac{1.41}{2}$ $dc = 3.33$ in

$ds := dc$

$A1 := ds \cdot 2 \frac{(CapWidth \cdot 12)}{BarNo}$ $A1 = 43.29$ in^2

$z := 170$ kip/in

$fs1 := \frac{z}{\sqrt[3]{dc \cdot A1}}$ $fs1 = 32.422$ ksi

A1 is Effective area
Z is crack width parameter

$fs2 := 0.6fy$ $fs2 = 36$ ksi

$fs := if(fs1 \le fs2, fs1, fs2)$ $fs = 32.422$ ksi

The smaller of the fs1 or fs2

$n := \frac{Es}{Ec}$ $n = 8.388$

LRFD 5.7.1
Repeat inf.
$As = 9.36$ top
$AsB = 7.8$ bottom

$p := \frac{As}{b \cdot d}$ $p = 6.728 \times 10^{-3}$

$k := -(p \cdot n) + \sqrt{(2 \cdot p \cdot n) + (p \cdot n)^2}$ $k = 0.284$

LRFD 5.7.3.1

$j := 1 - \frac{k}{3}$ $j = 0.905$

$AllowMs := As \cdot d \cdot j \cdot \frac{fs}{12}$ $AllowMs = 816.593$ kip ft

From Cap 18 output

$negServ = 1.064 \times 10^3$

Check to see if allowable is greater than Service Stress

$ServiceabilityMom := if[(AllowMs \ge negServ), OK, NG]$

$ServiceabilityMom = "NG"$

Check Serviceability Bottom

Control of cracking by distribution reinforcement LRFD 5.7.3.4
Clear Cover is 2 inches or less

dc is cover offer extreme tension fiber

ds is the centroid of the tensile reinfocement. For one steel layer dc=ds.

$dc := 2 + \left(\frac{5}{8}\right) + \frac{1.41}{2}$ $dc = 3.33$ in

$ds := dc$

$A1B := ds \cdot 2 \frac{(CapWidth \cdot 12)}{BarNoB}$ $A1B = 51.948$ in^2

A1 is Effective area

$z := 170$ kip/in — Z is crack control

$fs1B := \frac{z}{\sqrt[3]{dc \cdot A1B}}$ $fs1B = 30.51$ ksi

$fs2 := 0.6fy$ $fs2 = 36$ ksi

$fsB := if(fs1B \le fs2, fs1B, fs2)$ $\boxed{fsB = 30.51}$ ksi — The smaller of the fs1 or fs2

$n := \frac{Es}{Ec}$ $n = 8.388$ — LRFD 5.7.1

$pB := \frac{AsB}{b \cdot d}$ $pB = 5.607 \times 10^{-3}$

$kB := -(pB \cdot n) + \sqrt{(2 \cdot pB \cdot n) + (pB \cdot n)^2}$ $kB = 0.263$

$jB := 1 - \frac{kB}{3}$ $\boxed{jB = 0.912}$

Repeat information

$AsB = 7.8$ $posServ = 799.5$

$d = 35.67$

$jB = 0.912$ $posUlt = 1.182 \times 10^3$

$AllowMs := AsB \cdot d \cdot jB \cdot \frac{fsB}{12}$ $AllowMs = 645.318$ kip ft

$ServiceabilityMomB := if[(AllowMs \ge posServ), OK, NG]$

$\boxed{ServiceabilityMomB = \text{"NG"}}$

Check to see if allowable is greater than Service Stress

Check Dead Load Positive Moment

Check Mdl: $fdl := 22$ ksi

Tx DOT Limits the dead load to 22 ksi due to observed cracking under dead load

$AllowMdlp := AsB \cdot d \cdot jB \cdot \frac{fdl}{12}$ $AllowMdlp = 465.32$ kip ft

$DeadLoadMoment := if[(AllowMdlp \ge posDL), OK, NG]$

$posDL = 365$ repeat inf.

$\boxed{DeadLoadMoment = \text{"OK"}}$

Check Dead Load Negative Moment

Top Steel

Check Mdln: $fdl := 22$ ksi

Tx DOT Limits the dead load to 22 ksi due to observed cracking under dead load

$AllowMdln := As \cdot d \cdot j \cdot \frac{fdl}{12}$ $AllowMdln = 554.102$ kip ft

$DeadLoadMoment := if[(AllowMdln \ge negDL), OK, NG]$

$\boxed{DeadLoadMoment = \text{"NG"}}$

negDL is the moment due to dead load from cap 18 output

Flexural Steel Summary

$BarNo = 6$ Top Size #11

$BarNoB = 5$ Bottom Size #11

TxDOT does not use symmetrical flexural reinforcement to simplify placement and checking of steel in the field for the Type IV Bents .

Shrinkage LRFD 3.12.4

Due to the symmetry of the bridge superstructure, no forces are developed at the intermediate bend due to shrinkage of the superstructure.

Skin Reinforcment 5.7.3.4

Skin Reinforcement provided

$NoOfSkinBars := 5 \quad AreaNo5 := .31 \quad Dia5 := .625 \text{ in}$

$Ask := NoOfSkinBars \cdot AreaNo5 \quad Ask = 1.55 \text{ in}^2$

$Cover := 2.25 \text{ in} \quad Dia11 := 1.41 \text{ in}$

Ask per face required

$AskMin := 0.0120(d - 30) \quad AskMin = 0.068 \text{ in}^2$

$TensionSteel := if(As \geq AsB, As, AsB)$

$TensionSteel = 9.36$

Only one set of Tension reinforcement at at time, top or bottom and only the skin reinforcement that is in the tension zone

$$MinSkin := if\left(Ask \leq \frac{TensionSteel}{4}, OK, NG\right)$$

MinSkin = "OK"

Flexural depth de taken as the distance from the compression face to the centroid of the steel, positive moment region (in).

$de := d \quad de = 35.67 \text{ in}$

$$MaxSkSp := \frac{de}{6} \quad MaxSkSp = 5.945 \text{ in}$$

Check Max spacing de/6 or 12 in

$$SkinSpProv := \frac{[(CapDepth \cdot 12) - (Cover \cdot 2 + Dia5 \cdot 2 + Dia11 \cdot 2)]}{NoOfSkinBars + 1}$$

$SkinSpProv = 5.072 \text{ in}$

$SkinSpace := if[(SkinSpProv \leq MaxSkSp), OK, NG]$

Check Spacing of Skin Reinforcement

SkinSpace = "OK"

Shear Design (LRFD 5.8)

Flow Chart design procedure see Figure C 5.8.3.4.2-5

$\beta := 2$
$\theta := 45$ $\quad \phi v := 0.90 \quad bv := b \quad bv = 39 \text{ in}$

LRFD 5.8.3.4 β is 2, θ is 45 deg and α is 90

LRFD 5.8.3.3-1&2

Vn must be the lesser of Vc + Vs +Vp or 0.25*fc*bv*dv

LRFD 5.5.4.2.1 Values of ϕ

$Vs = (A_v*f_y*d_v*(\cot\theta + \cot\alpha)*\sin\alpha)*1/S$

For θ = 45 and α =90 it reduces to

$$Vs = A_v d_v f_y / S$$

$Vp := 0$ Prestress

Cantilever Section

$Vu := 520.0kips$ Sta 81 From Cap 18 output

Av=two legs of #5 shear steel
Sp=space of stirrups

$Sp := 6.0$ in $Av := .62$ in^2

$Avmin := 0.0316 \cdot Sp\sqrt{fc} \cdot bv \cdot \frac{1}{fy}$ $Avmin = 0.234$ in^2

$Avprovided := if[(Av \geq Avmin), OK, NG]$

$Avprovided = "OK"$

$Mn = 1.871 \times 10^4$ kip in with BarNo = 6

Find effective Shear Depth dv, dv1 must be greater than dv2 and dv3

$dv1 := \frac{Mn}{As \cdot fy}$ $dv1 = 33.317$ in

$dv2 := 0.9d$ $dv2 = 32.103$ in

$dv3 := 0.72 \cdot (CapDepth \cdot 12)$ $dv3 = 28.08$ in

$tempdv := if[(dv2 \geq dv3), dv1, dv3]$

$dv := if(dv1 \geq tempdv, dv1, tempdv)$

$dv = 33.317$ in

Effective Shear Depth

$Vc := 0.0316 \cdot \beta \sqrt{fc} \cdot bv \cdot dv$ $Vc = 155.812$ kips

LRFD 5.8.3.3.3-3 Vc shear resistance of the Concrete

$Vnmax := 0.25 \cdot fc \cdot bv \cdot dv$

$Vnmax = 1.169 \times 10^3$ kips

LRFD C 5.8.3.3-4 Tensile Stresses in transverse reinforcement {stirrups}

$Vs := Av \cdot dv \cdot \frac{fy}{Sp}$ $Vs = 206.566$ kips

$Vn := Vc + Vs$ $Vn = 362.377$ kips

Beta β is the factor that relates longitudinal strain on sheer capacity.

$Vn := if[(Vnmax \leq Vn), Vnmax, Vn]$

$Vn = 362.377$ cips

Find Max Vn nominal shear resistance LRFD 5.8.2.1

$Vr := \phi v \cdot Vn$ $Vr = 326.14$ kips

$MaxVrCL := if[(Vr \geq Vu), OK, NG]$

Check for Max shear on the cantilever section

$MaxVrCL = "NG"$

Increase Cap Depth as Necessary to satisfy this equation.

$vu := \frac{Vu - \phi v \cdot Vp}{\phi v \cdot b \cdot dv}$ $vu = 0.445$ ksi

Use to check maximum spacing

$Smax := if[(vu \geq 0.125 \cdot fc), if[(0.4 \cdot dv < 12), 0.4 \cdot dv, 12], if[(0.8 \cdot dv < 24), 0.8 \cdot dv, 24]]$

$\boxed{Smax = 24}$ in

$Sprovided := if[(Smax \geq Sp), OK, NG]$

$\boxed{Sprovided = "OK"}$

Section 1

$Vu1 := 446.($ at Sta 21 **Cap18 output**

Bottom steel section

$Av1 := .62\ in^2 \qquad Sp1 := 12$ in

$Avprovided := if[(Av1 \geq Avmin), OK, NG]$

$\boxed{Avprovided = "OK"}$

$MnB = 1.578 \times 10^4$ kip in

$dv1B := \frac{MnB}{AsB \cdot fy} \qquad dv1B = 33.709$ in

$dv2 := 0.9d \qquad dv2 = 32.103$ in

$dv3 := 0.72 \cdot (CapDepth \cdot 12) \qquad dv3 = 28.08$ in

$tempdvB := if[(dv2 \geq dv3), dv1B, dv3]$

$dvB := if(dv1B \geq tempdvB, dv1B, tempdvB)$

$dvB = 33.709$ in

$Vc := 0.0316 \cdot \beta \sqrt{fc} \cdot bv \cdot dvB \qquad Vc = 157.646$ kips

$VnmaxB := 0.25 \cdot fc \cdot bv \cdot dvB$

$VnmaxB = 1.183 \times 10^3$ kips

$Vs1 := Av1 \cdot dvB \cdot \frac{fy}{Sp1} \qquad Vs1 = 104.499$ kips

$VnS1 := Vc + Vs1 \qquad VnS1 = 262.14\text{4}$ kips

$VnS1 := if[(Vnmax \leq VnS1), Vnmax, VnS1]$

Find Max Vn nominal shear resistance for Section 1 LRFD 5.8.2.1

$VnS1 = 262.144$ kips

LRFD 5.8.2.1-2

$VrS1 := \phi v \cdot VnS1$

$VrS1 = 235.93$ kips

Check for Max shear on section 1

$MaxVr1 := if[(VrS1 \geq Vu1), OK, NG]$

$\boxed{MaxVr1 = "NG"}$

Maximum Nominal shear resistance Vr1 must be greater than Shear on the section Vu

$vu := \frac{Vu1 - (\phi v \cdot Vp)}{\phi v \cdot b \cdot dv} \qquad vu = 0.381$ ksi

$Smax := if[(vu \geq 0.125 \cdot fc), if[(0.4 \cdot dv < 12), 0.4 \cdot dv, 12], if[(0.8 \cdot dv < 24), 0.8 \cdot dv, 24]]$

$Smax = 24$ in

$Sprovided := if[(Smax \geq Sp1), OK, NG]$

$Sprovided = \text{"OK"}$

Section 2

$Vu2 := 563.8$ Sta 45 **Cap18 output**

$Av2 := 1.24$ in^2 $Sp2 := 4.8$ in

$Avprovided := if[(Av2 \geq Avmin), OK, NG]$

$Avprovided = \text{"OK"}$

$dv = 33.317$ in $Vnmax = 1.169 \times 10^3$ kips

Use dv based on negative moment

$Vs2 := Av2 \cdot dv \cdot \frac{fy}{Sp2}$ $Vs2 = 516.414$ kips

$VnS2 := Vc + Vs2$ $VnS2 = 674.06$ kips

$VnS2 := if[(Vnmax \leq VnS2), Vnmax, VnS2]$

$VnS2 = 674.06$ kips

Find Max Vn nominal shear resistance for Section 2 LRFD 5.8.2.1

$VrS2 := \phi v \cdot VnS2$

LRFD 5.8.2.1-2

$VrS2 = 606.654$ kips

$MaxVr2 := if[(VrS2 \geq Vu2), OK, NG]$

Check for Max shear on section 2

$MaxVr2 = \text{"OK"}$

$vu := \frac{Vu2 - (\phi v \cdot Vp)}{\phi v \cdot b \cdot dv}$ $vu = 0.482$ ksi

LRFD 5.8.2.9 Shear Stress

$Smax := if[(vu \geq 0.125 \cdot fc), if[(0.4 \cdot dv < 12), 0.4 \cdot dv, 12], if[(0.8 \cdot dv < 24), 0.8 \cdot dv, 24]]$

$Smax = 12$ in

$Sprovided := if[(Smax \geq Sp2), OK, NG]$

$Sprovided = \text{"OK"}$