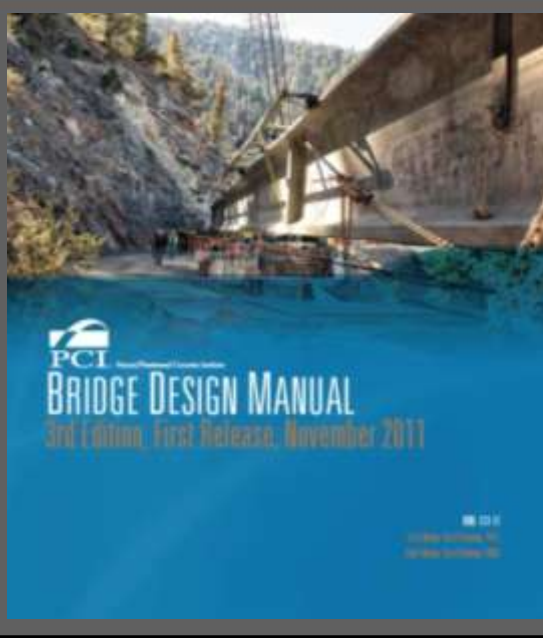
INTRODUCING: THE NEW PCI BRIDGE DESIGN MANUAL

William Nickas, PE
Mohsen Shahawy, PE
Dennis Mertz, PE



BDM HISTORY

- The original BDM dates to Oct. 1997.
  - At that time, many States still used the old AASHTO Standard Specifications.
- 2nd Edition 2003
- Between 1997 and 2009, chapters were added and existing chapter updated.
- Last Revision: June 2009 – added Chapters 5 (aesthetics) , 10 (bearings) and 20 (piles).
- Last update of Chapter 9, Design Examples was July 2003.
- Third Edition released 2011; through *LRFD Specifications, 5th Ed.*

WHY THE EXTENSIVE REWRITE?

- Standard Specification references and examples are no longer needed.
- The BDM needed to add changes in knowledge, technology and materials since the original version was written. This could not be done with a simple update.
- The BDM needed to be updated to current LRFD Specifications
- Ch 18 updated to include LRFR.

OUR GOAL

The Bridge Design Manual should be educational as well as an excellent reference on bridge design.

EDUCATION

- Expanded design examples show various options for bridge design methods
- Improved chapters cover complex or less common design methods.
- Information on new technologies

AUTHORS

- John Corven, PE
- John Dick
- Kevin Eisenbeis, PE
- Emily Lorenz, PE, LEED
- Dennis Mertz, PhD, PE
- William Nickas, PE

AUTHORS

- Henry Russell, PhD, PE
- Frank Russo, PhD, PE
- Moshen Shahawy, PhD, PE
- John Stanton, PhD
- Chuanbing Sun, PhD, PE
- Maher Tadros, PhD, PE

BLUE RIBBON PANEL

- S. Badie
- R. Brice
- H. Bonstedt
- R. Castrodale
- V. Chandra
- D. Chapman
- R. Cox
- R. Dorsife
- R. Eriksson
- J. Fabinski
- R. Frosh
- J. Grafton
- B. Greybeal
- R. Holt
- A. Keenan
- M. McCool

BLUE RIBBON PANEL

- R. Miller
- C. Prussack
- K. Roberts-Wollmann
- H. Ronald
- S. Seguirant
- B. Shivakumar
- C. Sun
- M. Tadros
- E. Wasserman
- C. White
- S. Zendegui
- T. Zokaie

GENERAL CHANGES TO ALL CHAPTERS

- Unnecessary material on Standard Specifications has been removed.
- All references to AASHTO or ASTM Specifications have been updated through 2011.
- Notation has been standardized and made consistent with all applicable AASHTO Specifications.

CHAPTER 1 - SUSTAINABILITY

A NEW CHAPTER!

ADDRESSES THE ISSUES/QUESTIONS ABOUT SUSTAINABILITY

CHAPTER 1 - SUSTAINABILITY

1.1 Scope

1.2 Life Cycle

– Addresses life cycle cost, service life and environmental assessment

1.3 Sustainability Concepts

– Triple bottom line, cost of green, “reduce, reuse, recycle”.

CHAPTER 1 - SUSTAINABILITY

1.4 Sustainability and Precast Concrete Bridges

– Durability, resistance to disasters and environmental benefits.

1.5 Sustainable Features of Precast Concrete

– Use of recycled/waste materials, use of local materials and reduction of waste in the factory.

CHAPTER 1 - SUSTAINABILITY

1.6 Simplified Tools and Rating Systems

1.7 State of the Art and Best Practices

- -Green plants are a reality at PCI
- -Second Generation of Plant Certification WILL have requirements for green plants (more to come!)

CHAPTER 2 - MATERIALS

2.1 Scope

2.2 Plant Products

2.3 Concrete Materials

2.4 Selection of Concrete Mix Requirements

2.5 Concrete Properties

2.6 Grout Materials

CHAPTER 2 - MATERIALS

2.7 Prestressing Strand

2.8 Non-Prestressed Reinforcement

2.9 Post-Tensioning Materials

2.10 Fiber Reinforced Plastic Reinforcement

2.11 Reinforcement Sizes and Properties

2.12 Relevant Standards and Publications

CHAPTER 2 - MATERIALS

WHAT'S NEW??

- Updated Material Standards
  - Ex. Updated M 240 – Blended Cements
- Inclusion of ASTM C1157 – Performance Based Specifications
- Information on ASR and DEF added

CHAPTER 2 - MATERIALS

- Expansion of HPC
  - High Strength
  - Low Permeability
  - Self Consolidating
  - Ultra High Performance
- Addresses the 11 elements of HPC
  - 4 on durability
  - 4 on strength
  - 3 on other properties

CHAPTER 3 - PRODUCTION

3.1 Scope
3.2 Product Components and Details
3.3 Fabrication
3.4 Plant Quality Assurance and Quality Control
3.5 Transportation
3.6 Installation
3.7 Diaphragms
3.8 Precast Deck Panels
3.9 References

CHAPTER 3 - PRODUCTION

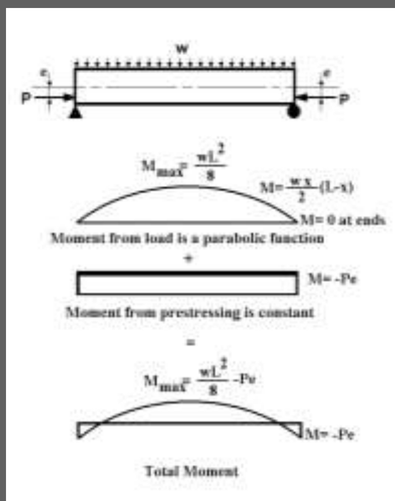
There are no major changes to Chapter 3.

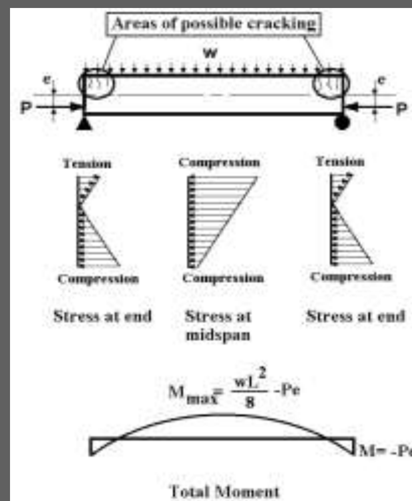
References have been added to:
-FHWA Report on Lightweight Concrete
–FHWA Report on UHPC Connections
-PCI Full Depth Deck Panel Report
-PCI State of the Art of Report on Box Girders

STRESSES AT TRANSFER OF PRESTRESSING FORCE

Based on a CHAPTER 9 –
DESIGN EXAMPLES

Stresses at release:





Allowable Stresses at Transfer of Prestressing Force

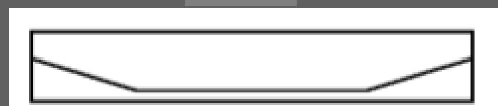
	Precast Beam
Compression	0.6 f_c' = 0.6(5.800 ksi) = +3.480 ksi
Tension with out bonded auxiliary reinforcement	0.0948$\sqrt{f_c'}$<0.200 ksi = -0.0948$\sqrt{5.800}$ = -0.228 ksi Therefore -0.200 ksi controls
Tension with bonded auxiliary reinforcement sufficient to resist 120% of the tension force in the cracked concrete.	-0.24$\sqrt{f_c'}$ = -0.24$\sqrt{5.800}$ = -0.578 ksi

(+) indicates compression; (-) indicates tension

IF AASHTO does not allow end cracking, how is end cracking avoided?

Debond

Harp

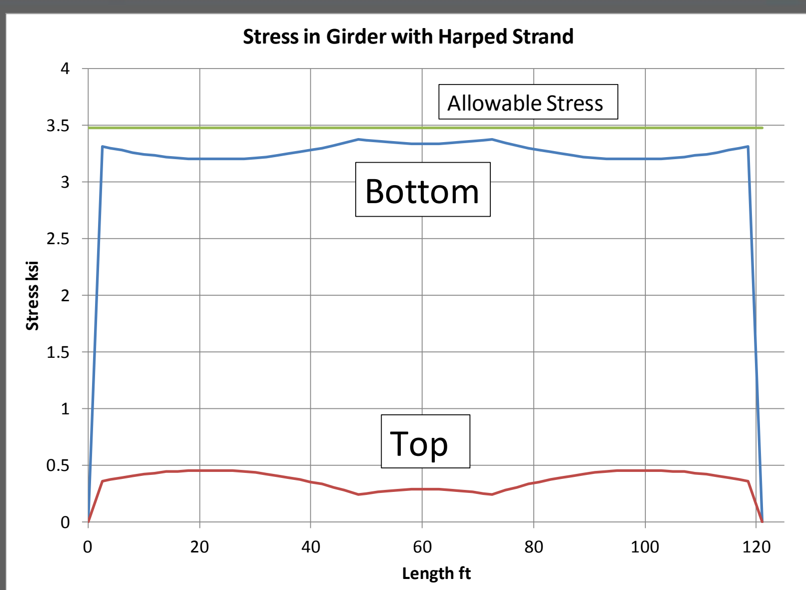


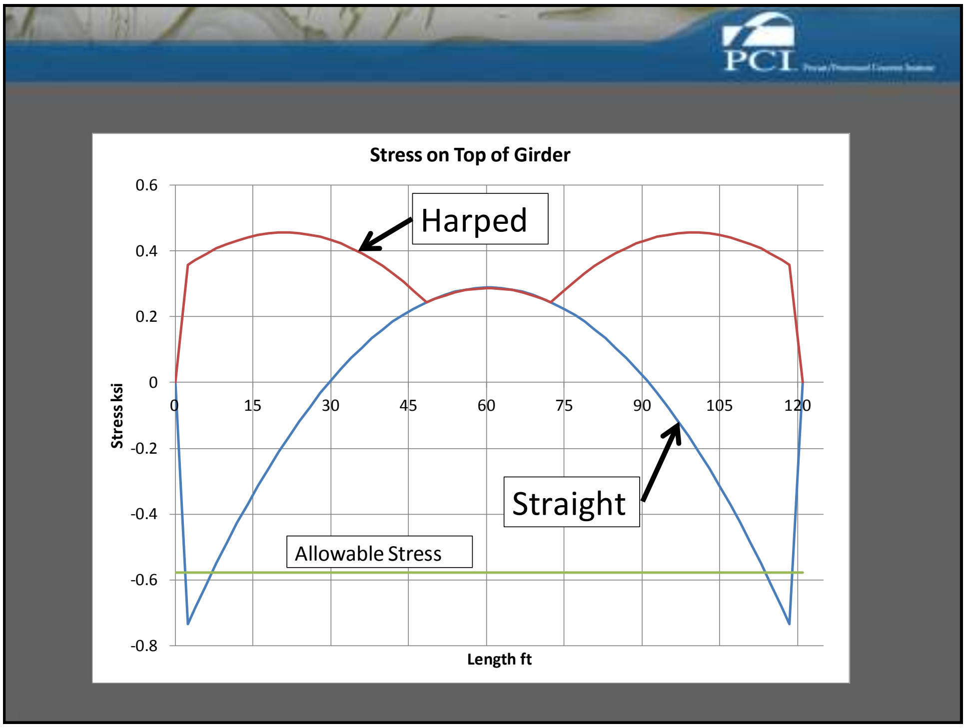
Harp or Debond??

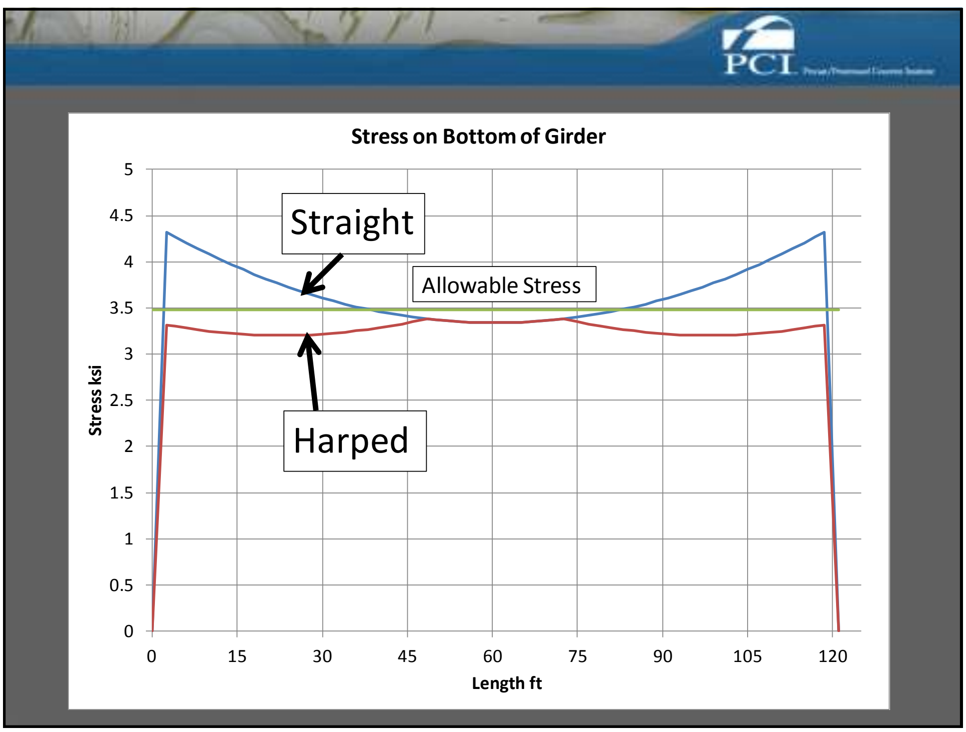
- Harping is not possible for certain sections, such as boxes.
- Always check State DOT practices. Some States prefer or require one practice over another.
- Check with local fabricators. Some fabricators do not have beds capable of handling uplift forces from harped strand.

HARPING

- Results in a more even stress distribution along the length of the girder.
- Vertical component of force helps resist shear.
- No effect on development length
- Holddown forces are high and not all beds can take it.
- Fabrication is more difficult.




PCI
Stress on Top of Girder
Harped
Straight
Allowable Stress
Stress ksi
Length ft
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
0
15
30
45
60
75
90
105
120

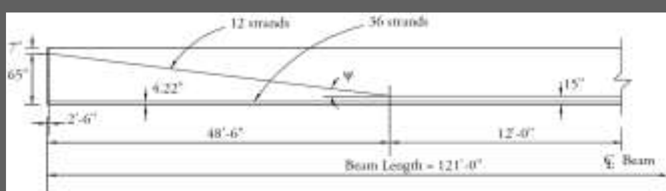

PCI
Stress on Bottom of Girder
Straight
Allowable Stress
Harped
Stress ksi
Length ft
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
15
30
45
60
75
90
105
120

Harping

The hold down force for the harped strands is now calculated. This would be checked with local practice to assure it does not exceed the capacity of the bed.

Assume the maximum strand stress will be:

$0.8f_{pu}$ = 0.8(270ksi) = 216 ksi
P_{harp} = 12 strand(216ksi)(0.153in^2)=397kips



$$\Psi = \text{Arc}\tan\left(\frac{(72-7-15)\text{in}}{48.5\text{ft}(12)}\right) = 4.9^0$$

$$F = 1.05(397\text{k})\sin\left(4.9^0\right) = 35.6\text{k}$$

The 1.05 accounts for friction in the hold down device.

CHAPTER 4 - ECONOMY

CHAPTER 4 - ECONOMY

Chapter 4 now discusses proper width of bearing pads, and refers the reader to Chapter 10.

CHAPTER 4 - ECONOMY

There are no major changes to Chapter 4.

Added references on:
-FHWA “Everyday Counts”
-FHWA Accelerated Bridge Construction
-NCHRP Reports 472 and 698 (seismic)

CHAPTER 5 - AESTHETICS

5.0 Introduction
5.2 Aesthetic Design Concepts
5.3 Project Aesthetics
5.4 Component Aesthetics
5.5 Appurtenance Aesthetics
5.6 Maintenance of Aesthetic Features
5.7 Cost of Aesthetics
5.8 Summary
5.9 Publications For Further Study

CHAPTER 5 - AESTHETICS

There are no changes to Chapter 5.

CHAPTER 6 – PRELIMINARY DESIGN

6.0 Scope

6.1 Preliminary Plan

6.2 Geometry

6.3 Substructures

6.4 Foundations

6.5 Preliminary Member Selection

6.6 Description of Design Charts

6.7 Preliminary Design Examples

6.8 References

Table 6.9-1
Design Charts

Chart No.	Beam Type	Chart Type
BB-1	AASHTO Box Beams 48 in. Wide	Maximum span versus beam spacing
BB-2	AASHTO Adjacent Box Beams 48 in. Wide	No. of strands versus span length
BB-3	AASHTO Spread Box Beams BII-48	No. of strands versus span length
BB-4	AASHTO Spread Box Beams BIII-48	No. of strands versus span length
BB-5	AASHTO Spread Box Beams BIV-48	No. of strands versus span length
BB-6	AASHTO Box Beams 36 in. Wide	Maximum span versus beam spacing
BB-7	AASHTO Adjacent Box Beams 36 in. Wide	No. of strands versus span length
BB-8	AASHTO Spread Box Beams BII-36	No. of strands versus span length
BB-9	AASHTO Spread Box Beams BIII-36	No. of strands versus span length
BB-10	AASHTO Spread Box Beams BIV-36	No. of strands versus span length

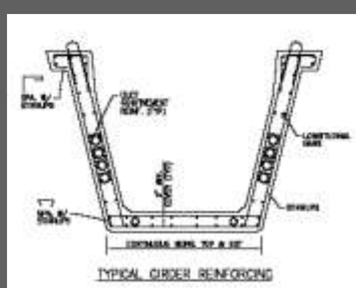
Table 6.9-1Design Charts

Chart No.	Beam Type	Chart Type
BT-1	AASHTO-PCI Bulb-Tees	Maximum span versus beam spacing
BT-2	AASHTO-PCI Bulb-Tees BT-54	No. of strands versus span length
BT-3	AASHTO-PCI Bulb-Tees BT-63	No. of strands versus span length
BT-4	AASHTO-PCI Bulb-Tees BT-72	No. of strands versus span length
DBT-1	Deck Bulb-Tees	Maximum span versus section depth
DBT-2	Deck Bulb-Tees	No. of strands versus span length
IB-1	AASHTO I-Beams	Maximum span versus beam spacing
IB-2	AASHTO I-Beams Type II	No. of strands versus span length
IB-3	AASHTO I-Beams Type III	No. of strands versus span length
IB-4	AASHTO I-Beams Type IV	No. of strands versus span length
IB-5	AASHTO I-Beams Type V	No. of strands versus span length
IB-6	AASHTO I-Beams Type VI	No. of strands versus span length

Table 6.9-1
Design Charts

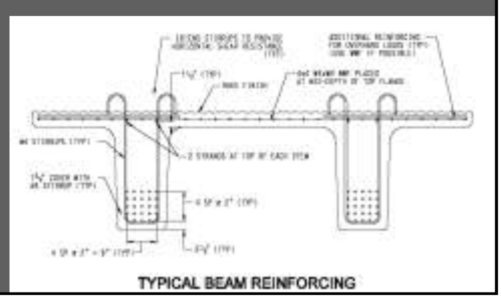
Chart No.	Beam Type	Chart Type
NEXT-1	NEXT Type D Beams	Maximum span versus section depth
NEXT-2	NEXT Type D x 96 Beams	No. of strands versus span length
NEXT-3	NEXT Type D x 120 Beams	No. of strands versus span length
NEXT-4	NEXT Type F Beams	Maximum span versus section depth
NEXT-5	Next Type F x 96 Beams	No. of strands versus span length
NEXT-6	Next Type F x 144 Beams	No. of strands versus span length
U-1	U-Beams	Maximum span versus beam spacing
U-2	Texas U-40 Beams	No. of strands versus span length
U-3	Texas U-54 Beams	No. of strands versus span length
U-4	Washington U66G5Beams	No. of strands versus span length
U-5	Washington U78G5 Beams	No. of strands versus span length

CHAPTER 6 – PRELIMINARY DESIGN

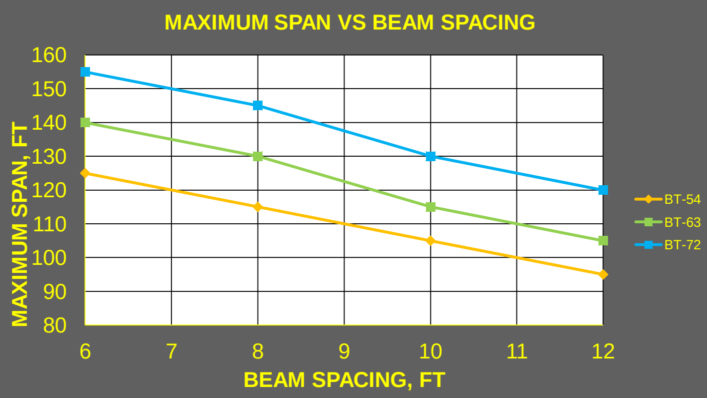


U Beam

NEXT Beam

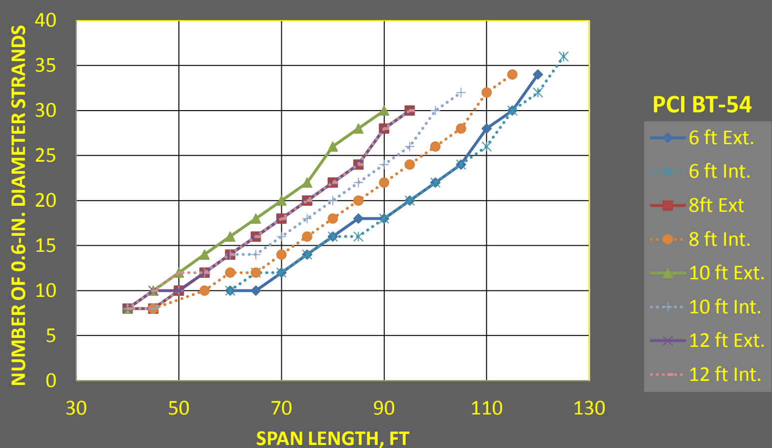


CHAPTER 6 – PRELIMINARY DESIGN



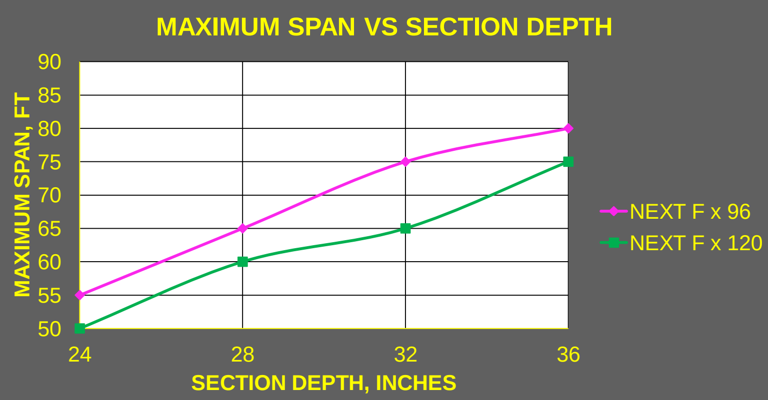
Maximum Spans for Bulb Tee Girders

CHAPTER 6 – PRELIMINARY DESIGN



Number of Strands vs. Span Length for Bulb Tee Girders

CHAPTER 6 – PRELIMINARY DESIGN



Graph for NEXT Type F Beams

CHAPTER 6 - PRELIMINARY DESIGN

Table BB-2
AASHTO Adjacent Box Beams 48 in. Wide

Spacing ft	Span ft	Slab Thickness in.	f'_{ci} ksi	No. of Strands	Camber in.	f_b @ $L/2$ ksi	f_t @ $L/2$ ksi	M_u @ $L/2$ ft-kips	M_r @ $L/2$ ft-kips	Control
AASHTO BII Adjacent 48-in.-Wide Exterior Box Beam										
BII	40	6	1.358	6	0.08	0.059	0.454	817	1,077	Strength
BII	45	6	1.344	6	-0.02	-0.121	0.610	992	1,077	Strength
BII	50	6	1.813	8	0.03	-0.053	0.720	1,186	1,414	Strength
BII	55	6	1.800	8	-0.18	-0.269	0.910	1,393	1,414	Strength
BII	60	6	2.266	10	-0.18	-0.238	1.051	1,612	1,741	Strength
BII	65	6	2.727	12	-0.21	-0.229	1.208	1,843	2,058	Strength
BII	70	6	3.185	14	-0.27	-0.240	1.382	2,088	2,365	Strength
BII	75	6	3.178	14	-0.87	-0.517	1.631	2,345	2,365	Stress
BII	80	6	4.091	18	-0.58	-0.326	1.779	2,615	2,951	Stress
BII	85	6	4.540	20	-0.87	-0.399	2.001	2,898	3,231	Stress
BII	90	6	4.986	22	-1.26	-0.493	2.240	3,194	3,502	Stress
BII	95	6	5.612	25	-1.54	-0.517	2.490	3,503	3,873	Stress
BII	100	6	6.409	29	-1.65	-0.479	2.754	3,825	4,327	Stress

CHAPTER 7LOADS & LOAD DISTRIBUTION
CHAPTER 7- Loads and Load distribution
Chapter 8 – Design Theory
Chapter 9 – Design Examples

Changes by Dr. Mertz and Dr. Shahawy

CHAPTER 10 -BEARINGS

10.1 Introduction
10.2 History of Elastomeric Bearings
10.3 Specifications
10.4 Loads and Movements for Design
10.5 Planning the Bearing Layout
10.6 Types of Elastomeric Bearings
10.7 Behavior of Elastomeric Bearings

CHAPTER 10 -BEARINGS

10.8 Design of Elastomeric Bearings

Covers BOTH Method A And B

10.9 Bearing Selection Guide

10.10 References

CHAPTER 10 -BEARINGS

- Chapter 10 has been completely rewritten.
- Both Methods A and B for bearing design are covered.
  - Design examples of each method are provided.

CHAPTER 11 – EXTENDING SPANS

CHAPTER 11 – EXTENDING SPANS

CHAPTER 11 – EXTENDING SPANS

- Design example in Chapter 11 violates *AASHTO LRFD Specifications* duct to web thickness ratio.
  - This example was based on older designs where steel ducts could take the grout pressure.
  - Newer plastic ducts cannot take the pressure so larger webs are needed.

CHAPTER 12 – SKEWED AND CURVED BRIDGES

12.1 – Scope

12.2 - Skew And Grade Effects

12.3 - Curved Bridge Configurations

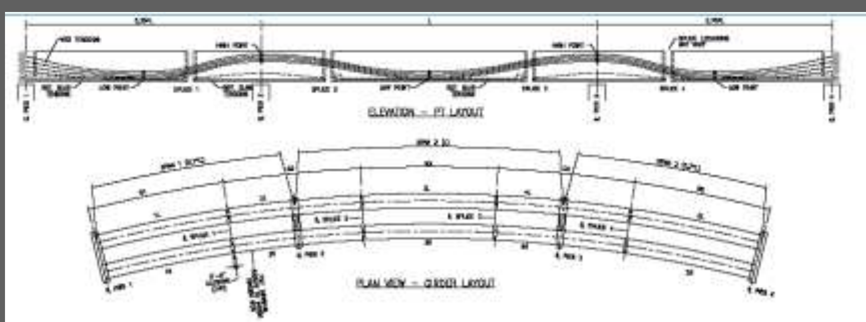
12.4 - Useful Geometric Approximations

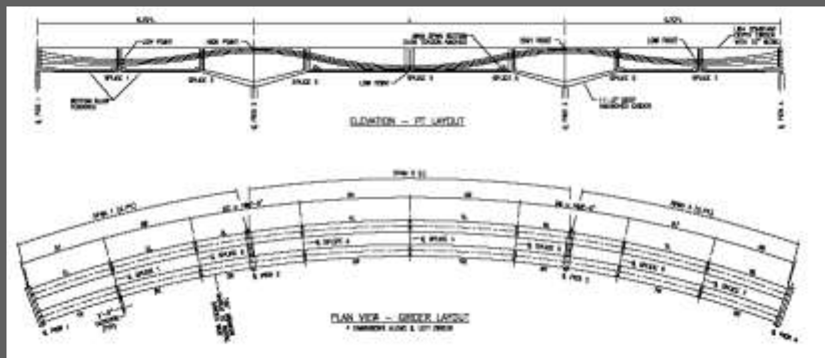
12.5 - Structural Behavior Of Curved Bridges

12.6 - Design Considerations

CHAPTER 12 – SKEWED AND CURVED BRIDGES

Chapter 12 was expanded to include the use of curved "U" beams. PCI Zone 6 Standard for curved "U" beams is shown.





CHAPTER 13 – INTEGRAL BRIDGES

CHAPTER 13 – INTEGRAL BRIDGES

13.8. - Case Studies Summary

13.9. - Conclusions

13.10. - References

13.11 - Bibliography

CHAPTER 14 – SEGMENTAL BRIDGES

14.1 INTRODUCTION

- 14.1.1 Balanced Cantilever Method
- 14.1.2 Span-by-Span Method

14.2 PRECAST SEGMENTS

14.3 TRANSVERSE ANALYSIS

- 14.3.1 Modeling for Transverse Analysis
- 14.3.2 Analysis for Uniformly Repeating Loads
- 14.3.3 Analysis for Concentrated Wheel Live Loads
- 14.3.4 Transverse Post-Tensioning

CHAPTER 14 – SEGMENTAL BRIDGES

14.4 Balanced Cantilever Construction
14.5 Span-by-span Construction
14.6 Diaphragms, Anchor Blocks And Deviation Details
14.7 Geometry Control
14.8 Prestressing With Post-tensioning
14.10 PCI Journal Segmental Bridge Bibliography

CHAPTER 15 - SEISMIC DESIGN

This chapter will be totally rewritten

Anticipated release in Fall 2012.

CHAPTER 16 – ADDITIONAL BRIDGE PRODUCTS

Expected last quarter 2012.

CHAPTER 17 – RAILROAD BRIDGES

This chapter has been rewritten to cover the current AREMA Manual.

Dr. Dennis Mertz

Chapter 7

Loads and Load Distribution

CHAPTER 7 – LOADS & LOAD DISTRIBUTION

7.1 Scope

7.2 Load Types

7.3 Load Combinations and Design Methods

7.4 Simplified Distribution Methods

7.5 Refined Analysis Methods

7.6 References

CHAPTER 7 – LOADS & LOAD DISTRIBUTION

Major Changes

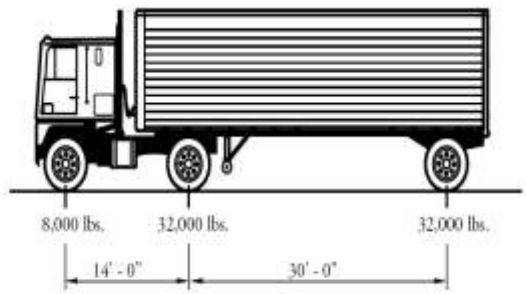
- More detailed information on fatigue.
- Information on loads in the *LRFD Specifications* has been updated.
- *Standard Specifications* information has been removed.

FATIGUE

Fatigue Analysis:

1) Uses a special "fatigue truck"
2) Does NOT use a lane load
3) Uses IM=15%
4) Uses one lane Distribution Factor
5) Does NOT use multiple presence factors. Approximate distribution factors include multiple presence factors, so the DF is divided by the multiple presence factor = 1.2
6) Has a load factor of 1.5

Fatigue Truck



Fatigue Truck

Fatigue Truck

LRFD Article 5.5.3.1 states that in fully prestressed components other than segmentally constructed bridges, the compressive stress due to Fatigue I load combination and one half the sum of effective prestress and permanent loads shall not exceed 0.40, after losses.

CHAPTER 8 – DESIGN THEORY & PROCEDURE

CHAPTER 8 – DESIGN THEORY & PROCEDURE

CHAPTER 8 – DESIGN THEORY & PROCEDURE

8.11 -Bending Moments and Shear Forces Due To Vehicular Live Loads

8.12 Strut-and-tie Modeling of Disturbed Regions

8.13 Detailed Methods of Time-dependent Analysis

8.14 References

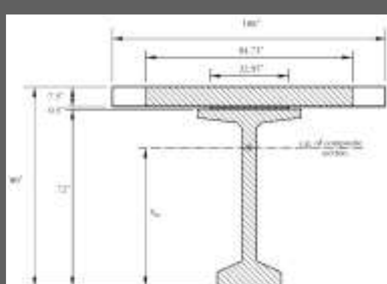
CHAPTER 8 – DESIGN THEORY & PROCEDURE

- Sectional and Simplified methods for shear resistance calculation are presented
  - Sectional Model (modified compression field theory) using equations for β and θ.
  - Simplified Method (V_{ci} and V_{cw})
- Updated Horizontal Shear provisions

CHAPTER 8 – DESIGN THEORY & PROCEDURE

- Addresses variability of camber between beams.
- Improved discussion of lateral stability.

CHAPTER 8 – DESIGN THEORY & PROCEDURE



GROSS COMPOSITE SECTION PROPERTIES

$$n = \frac{E_{slab}}{E_{Girder}} = \frac{3834ksi}{4888ksi} = 0.7845$$

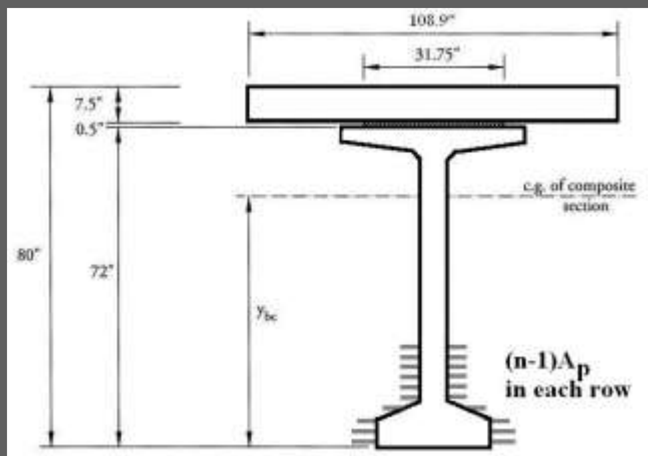
Table 9.1a.3.2.3-1
Properties of Composite Section

	Transformed Area, 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}
Beam	767.00	36.60	28,072	253,224	545,894	799,118
Haunch	16.47	72.25	1,190	5,032	0.34	5,032
Deck	635.45	76.25	48,453	293,191	2,979	296,170
∑	1,418.9		77,715			1,100,320

STOP If using gross properties!!

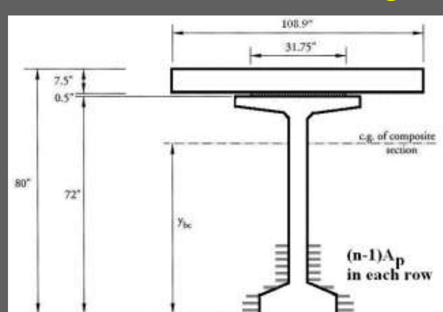
CHAPTER 8 – DESIGN THEORY & PROCEDURE

Calculation of Transformed Properties



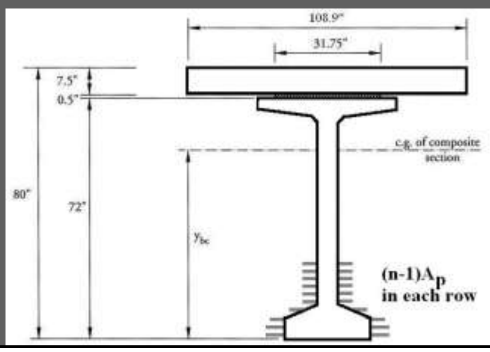
CHAPTER 8 – DESIGN THEORY & PROCEDURE

Transformed sections provide a more accurate service level stress calculation AND transformed sections implicitly account for elastic shortening losses!



CHAPTER 8 – DESIGN THEORY & PROCEDURE

AASHTO says that if transformed sections are used, ES is taken as “0”. This does not mean ES is ignored! Transformed section implicitly accounts for ES!



Note: ES still needs to be calculated for determining casting length.

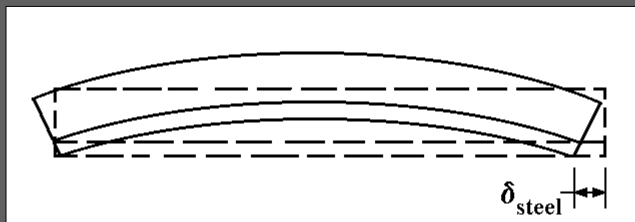
CHAPTER 8 – DESIGN THEORY & PROCEDURE

When finding elastic shortening, it is necessary to determine the effective force after transfer of prestressing force, P_p. It is NOT P_i, the prestressing force before transfer. AASHTO requires the engineer estimate P_p and iterate!

$$\Delta f_{pES} = \frac{E_p}{E_{ct}}\left(\frac{P_p}{A_g} + \frac{P_p e y}{I_g} - \frac{My}{I_g}\right)$$

$$P_p = P_i - A_{ps}\Delta f_{pES}$$

CHAPTER 8 – DESIGN THEORY & PROCEDURE

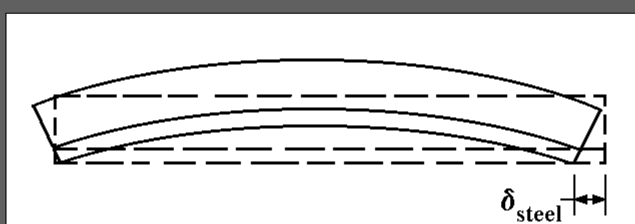


Look at how transformed section accounts for ES.

Consider a beam at transfer of the prestressing force.

The steel gets shorter and loses some stress.

CHAPTER 8 – DESIGN THEORY & PROCEDURE



The section shortens due to axial load and moment.

Elastic superposition applies.

CHAPTER 8 – DESIGN THEORY & PROCEDURE

So when the AASHTO Specifications say use ES=0 when using transformed section, it is NOT ignoring ES.

ES is implicit in the equations for stress when transformed sections are used!

CHAPTER 8 – DESIGN THEORY & PROCEDURE

Equation C5.9.5.2.3b-1 from the commentary is just the rearranged transformed section equation:

$$\Delta f_{pES} = \frac{A_{ps} f_{pi}(I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps}(I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

CHAPTER 8 – DESIGN THEORY & PROCEDURE

There are two choices for determining long term loss of prestressing force (LRFD Specifications 5.9.5):

- Approximate (5.9.5.3)
- Refined (5.9.5.4)

CHAPTER 8 – DESIGN THEORY & PROCEDURE

The approximate long term loss equations are shown here. Note that is it basically a "lump sum" approach and applies only to pretensioned members with cast in place decks under "normal" conditions.

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$

CHAPTER 8 – DESIGN THEORY & PROCEDURE

Why use the Refined Method for long term loss of prestressing force?

- Refined gives the state of stress at every important time step.
- Refined is required for post-tensioning.
- Refined is required for pretensioned without CIP decks.
- Refined is required for members which do not meet the conditions of 5.9.5.3 allowing the use of approximate method.
- The approximate method may overstate creep.
- Approximate MAY be used for piles.

CHAPTER 8 – DESIGN THEORY & PROCEDURE

This is the basic equation for the refined method:

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$$

CHAPTER 8 – DESIGN THEORY & PROCEDURE

Long term losses from transfer of prestressing force to casting the deck:

- Δf_{pSR} = prestress loss due to shrinkage of girder concrete between transfer of prestressing force and deck placement (ksi)
- Δf_{pCR} = prestress loss due to creep of girder concrete between transfer of prestressing force and deck placement (ksi)
- Δf_{pR1} = prestress loss due to relaxation of prestressing strands between transfer of prestressing force and deck placement (ksi)

CHAPTER 8 – DESIGN THEORY & PROCEDURE

Long term loss of prestressing force after the deck is cast:

- Δf_{pR2} = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (ksi)
- Δf_{pSD} = prestress loss due to shrinkage of girder concrete between time of deck placement and final time (ksi)
- Δf_{pCD} = prestress loss due to creep of girder concrete between time of deck placement and final time (ksi)
- Δf_{pSS} = prestress gain due to shrinkage of deck in composite section (ksi)

CHAPTER 8 – DESIGN THEORY & PROCEDURE

SAME type TOOLS:

The refined method (5.9.5.4.2a-1) calculates the potential shrinkage and then modifies it.

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id}$$

$$K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_c} \left(1 + \frac{A_c e_{pc}^2}{I_c}\right) \left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]}$$

K_{id} uses girder properties.

CHAPTER 8 – DESIGN THEORY & PROCEDURE

Incremental steps skipped to save time

Refined Long term loss:

- prestress loss due to relaxation
- prestress loss due to shrinkage of girder
- prestress loss due to creep of girder
- prestress gain due to shrinkage of deck

- Let's FOCUS on the GAIN due to Deck Shrinkage

CHAPTER 8 – DESIGN THEORY & PROCEDURE

Finally, prestressing force gain due to differential shrinkage of the deck:

$$\Delta f_{pSS} = \frac{E_p}{E_c} \Delta f_{cdf} K_{df} \left[1 + 0.7\Psi_b\left(t_f, t_d\right) \right]$$

$\Psi_b(t_f,t_d)$ is the creep coefficient for the girder calculated between the time of deck placement and the final time.

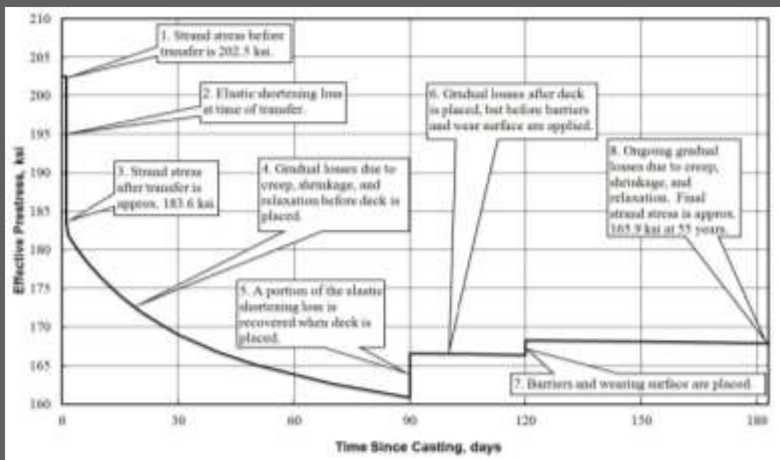
CHAPTER 8 – DESIGN THEORY & PROCEDURE

The change in concrete stress at the centroid of the prestressing steel due to deck shrinkage:

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{1 + 0.7\Psi_d\left(t_f, t_d\right)} \left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c} \right)$$

PCI

CHAPTER 8 – DESIGN THEORY PROCEDURE



PCI

CONCRETE STRESSES AT SERVICE LOADS

CHAPTER 9 –
DESIGN EXAMPLES

Allowable Service Level Stresses

	Precast Beam	Deck
Compression under permanent load	$0.45\ f_c' =$ 0.45(6.500) = +2.925 ksi	$0.45\ f_c' =$ 0.45(4.000) = +1.800 ksi
Compression under all loads	$0.60f_c' =$ 0.60(6.500) = +3.900 ksi	$0.60f_c' =$ 0.60(4.000) = +2.400 ksi
Tension	$-0.19\sqrt{f_c'}$ = -0.19√(6.500) = -0.484 ksi	N/A

CHAPTER 9 – DESIGN EXAMPLE 9.1a

Here is a summary of the stresses:

Stresses at Midspan at Service Loads

Design Example	Top of Deck, ksi Service I		Top of Beam, ksi Service I		Bottom of Beam, ksiService III
	Permanent Loads	Total Loads	Permanent Loads	Total Loads	
9.1a	+0.114	+0.677	+1.737	+2.237	+0.154
Allowable	+1.800	+2.400	+2.925	+3.900	-0.484

How do these values change if non-transformed properties are used?

First, because non-transformed properties are used, elastic stresses are no longer implicit, so elastic losses must be included.

Also, the loss values change when gross section properties are used. From Design Example 9.1b:

P_{pe} = 1232 kips

Reminder of Non-transformed Composite Properties:

A = 1418.9 in^2
y_{bc} = 54.77 in.
y_{tp} = 72-54.77 = 17.23 in. to top of precast
y_{tc} = 80-54.77 = 23.23 in. to top of composite
I = 1100320 in^4
e = 29.68 in (calculated for final strand pattern)

These properties were used to estimate the number of strands!

Previously, the stress due to applied loads was found:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_b + M_{ws} + 0.8M_{LL+I}}{S_{bc}}$$

$$f_b = \frac{((1{,}438.2+1{,}659.6))(12)}{14{,}915} + \frac{((180+360)+(0.8)(1{,}830.2+843.3))(12)}{20{,}090}$$

$$f_b = 2.492 + 1.600 = 4.092ksi$$

CHAPTER – DESIGN EXAMPLE 9.1a

$$f_b = \frac{1232k}{767in^2} + \frac{1232k(29.68in)}{14915in^3} - 4.092ksi$$

$$f_b = -0.034ksi$$

Using transformed sections, the stress was +0.154 ksi.

How do the stresses change if approximate long term losses are used?

$$\Delta f_{pLT} = 10\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + \Delta f_{pR}$$

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{1+f_{ci}'}$$

Using 70% RH

$$\gamma_h = 1.7 - 0.01H = 1.7 - 0.01(70) = 1$$

$$\gamma_{st} = \frac{5}{1+f_{ci}'} = \frac{5}{1+5.8} = 0.735$$

$$\Delta f_{pLT} = 10\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + \Delta f_{pR}$$

$$\Delta f_{pLT} = 10\frac{202.5ksi\left(7.34in^2\right)}{767in^2}(1)(0.735) + 12(1)(0.735) + 2.5ksi$$

$$\Delta f_{pLT} = 25.5ksi$$

EFFECT OF DECK SHRINKAGE !

According to limited research relative shrinkage between the CIP deck and precast girder could lead to additional tension stress in the girder bottom fibers

EFFECT OF DECK SHRINKAGE !

It is likely, however, that the full calculated force from deck shrinkage will not occur because of the presence of deck cracking and deck reinforcement. The following Example illustrates the theoretical effect of the deck shrinkage for the effect of applying 0, 50, or 100% of the calculated deck force on the stresses at load combination Service III.

EFFECT OF DECK SHRINKAGE !

Analyzing Deck shrinkage as an external force applied to the composite non transformed section yields below values when compared to a bottom stress of +0.154 ksi.

Table 9.1a.8.5.4-1
Stresses at Midspan for Load Combination Service III Including the Effect of Deck Shrinkage.

Deck Shrinkage Force, %	Bottom of Beam, ksi Service III
0	+0.127
+50	−0.001
100	−0.128

Service stress observations:

Many States have adopted an analysis method in their Manuals.

Calculated bottom fiber stresses depends:

- Based on loss method
- Method used for section properties
- Treatment of deck shrinkage.

Note: Careful attention is needed when applying gains to prestressed loss calculations!!!

Thank you for your attention.

Questions?

CHAPTER 9 –
DESIGN EXAMPLES

Strength Limit State

Use Strength I Load Combination

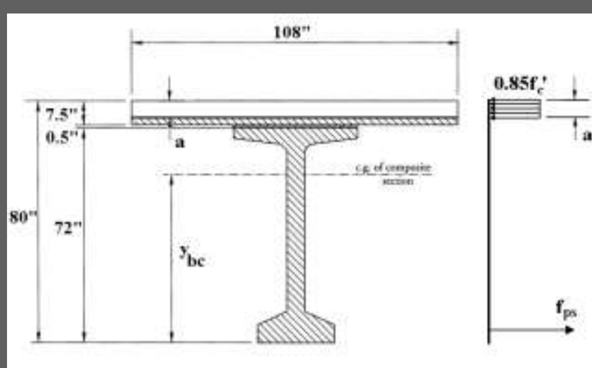
M_u = 1.25DC + 1.5DW + 1.75(LL+IM)

DC = Dead loads applied at construction

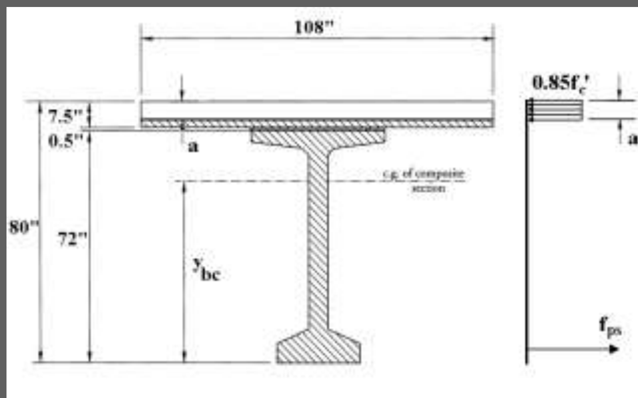
DW = Future wearing surfaces/utilities

LL+IM = Live load (with impact)

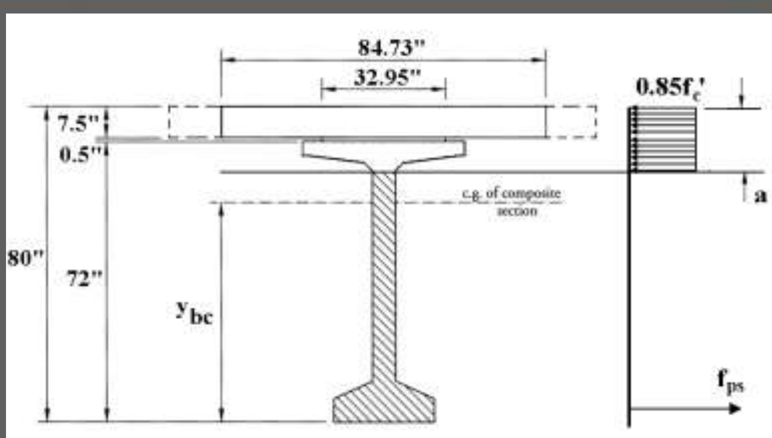
DESIGN EXAMPLE 9.1 a, b, c



Prestressed concrete uses the same method for finding M_n as reinforced concrete, except that the steel stress must be calculated.

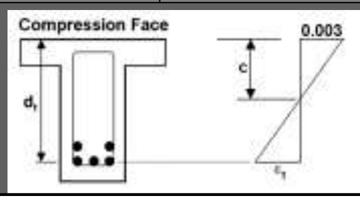


One possibility is the stress block falls in the slab. The actual slab properties would be used.



T-Beam behavior is also possible.

Strain in the extreme tensile steel, ε_t.	Type of Section
$\varepsilon_t \leq f_y/E_s$ $\varepsilon_t \leq 0.002$ for prestressed.	**Compression Controlled** $c/d_t \geq 3/5$*
$0.005 > \varepsilon_t > f_y/E_s$ $0.005 > \varepsilon_t > 0.002$ for prestressed	**Transition** $3/8 < c/d_t < 3/5$*
$\varepsilon_t > 0.005$	**Tension Controlled** $c/d_t \leq 3/8$



For a tension controlled section, $\Phi = 1$

$\Phi M_n > M_u$

This check needs to be made at all sections along the girder.

Ductility Limit (2012 AASHTO):

$\Phi M_n \geq M_{cr} \leq 1.33M_u$

This check needs to be made at all sections along the girder.

$$M_{cr} = \gamma_3 \left[\left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) S_{btc} - M_{dnc} \left(\frac{S_{btc}}{S_{btf}} - 1 \right) \right]$$

$$f_r = 0.24\sqrt{f_c'} \quad \textit{Modulus of rupture}$$

$$f_{cpe} = \frac{P_{pe}}{A_{tf}} + \frac{P_{pe} e_{tf}}{S_{btf}}$$

Ductility Limit continued (2012 AASHTO):

Definitions:

γ_1 = flexural cracking variability factor
- 1.2 for precast segmental
- 1.6 for all other cases

γ_2 = prestress variability factor
- 1.1 for bonded
- 1.0 for unbonded

γ_3 = ratio of specified yield strength to ultimate strength
- 0.67 for A615 GR 60
- 0.75 for A706 GR 60
- 1.00 for prestressed

SHEAR

The *Standard Specifications* used:

$$V_{ci} = 0.02\sqrt{f'_c}b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}b_v d_v$$

$$V_{cw} = \left(0.06\sqrt{f'_c} + 0.30 f_{pc}\right) b_v d_v + V_p$$

V_{ci} = flexural shear
V_{cw} = web shear

CHAPTER 9 – DESIGN EXAMPLES

The LRFD Specifications now allow:

- Shear design using the sectional model (5.8.3.4.2). This is based on Modified Compression Field Theory.
- Shear design using the simplified method (5.8.3.4.3). This is a modified version of V_{ci} and V_{cw}.
- Shear design using Appendix B. This is the sectional model using tables.

CHAPTER 9 – DESIGN EXAMPLES

Sectional Model (5.8.3.4.2)

- Based on Modified Compression Field Theory.
- Requires the calculation of β and θ for V_s and V_c. (α is stirrup angle)

$$V_c = 0.0316\,\beta\sqrt{f_c'}\;b_v d_v$$

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

CHAPTER 9 – DESIGN EXAMPLES

Sectional model requires finding the strain in the longitudinal steel:

$$\varepsilon_s = \frac{\left|\frac{M_u}{d_v}\right| + 0.5N_u + \left|V_u - V_p\right| - A_{ps}f_{po}}{\left(E_sA_s + E_pA_{ps}\right)}$$

$$\varepsilon_s = \frac{\left|\frac{M_u}{d_v}\right| + 0.5N_u + \left|V_u - V_p\right| - A_{ps}f_{po}}{\left(E_sA_s + E_pA_{ps} + E_cA_{ct}\right)} \geq -0.0004$$

Originally, $|V_u-V_p|$ was $2|V_u-V_p|\cot\theta$.

CHAPTER 9 – DESIGN EXAMPLES

Sectional Model (5.8.3.4.2)

- In the V1 of the *LRFD Specifications*, finding β and θ was iterative.
  - Critical section location was at $d_v\cot\theta$ but forces at the critical section were needed to find θ.
  - Finding θ required the calculation of the strain in the longitudinal steel, but this was also a function of θ.
- Finding β and θ required using a table.

CHAPTER 9 – DESIGN EXAMPLES

Modifications to Sectional Model (5.8.3.4.2)

- Critical section was simplified to the shear depth (d_v) from the face of the support.
- In finding the strain the in longitudinal steel, $\cot\theta$ was set at 0.5 to prevent iteration.
- Tables for β and θ were replaced with formulae.

CHAPTER 9 – DESIGN EXAMPLES

- Section Model now has equations for β and θ.
  - Easier to use the method.
  - Easier to program
- Appendix B of Chapter 5 still retains the old tables from previous versions of the LRFD Specifications.
- The BDM illustrates both methods.

CHAPTER 9 – DESIGN EXAMPLES

The Simplified Method (5.8.3.4.3)

- This is a modified version of the Standard Specifications equations.
  - These are still used in ACI 318.
- V_{ci} is the shear which causes a flexural crack to become a shear crack. This is flexural shear capacity.
- V_{cw} is the shear which causes a principal tensile stress of $4\sqrt{f_c'}$. This is web shear capacity.

CHAPTER 9 – DESIGN EXAMPLES

Why the Simplified Method (5.8.3.4.3) was added in 2007:

- It was a popular and preferred by some engineers for hand calculations.
- The original versions of Sectional Model were complex and iterative.
- NCHRP Report 549 recommended inclusion in the *LRFD Specifications.*
  - NCHRP suggested some modifications to this method which were adopted.

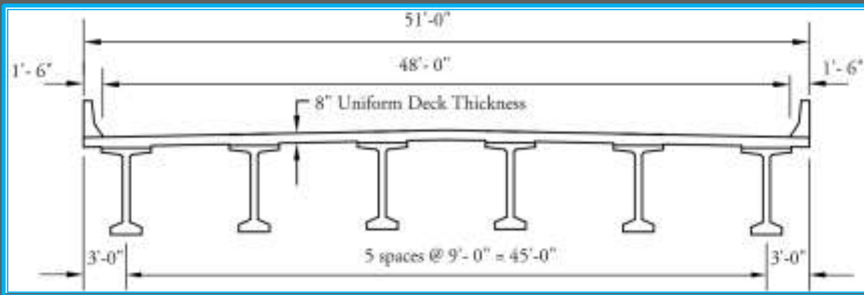
CHAPTER 9 – DESIGN EXAMPLES

Modifications for Simplified Method:

- The formula for V_{ci} requires subtracting out DL, but DL was never clearly defined. This was now defined as non-composite DL.
- The calculation of V_s requires cotθ. This was defined as 1 for V_{ci} and for V_{cw}:

$$\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \le 1.8$$

PCI EXAMPLE 9.1a, 9.1b, 9.1c



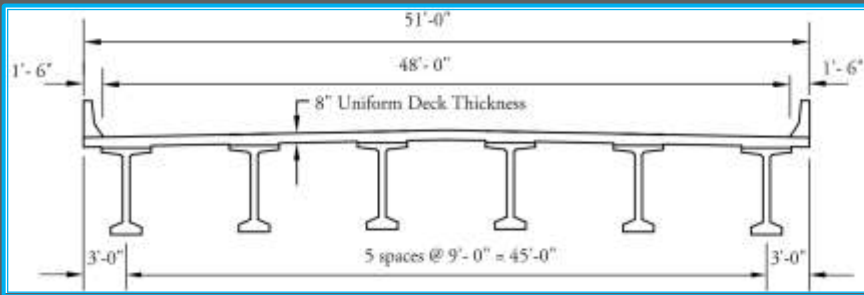
	Bridge Type	Span	Cross Section	Prestress Losses	Shear
9.1a	BT-72 beams with CIP composite deck	One simple span 120'	Transformed	Refined	General
9.1b			Gross	Refined	Appendix B5
9.1c			Transformed	Approximate	Simplified

CHAPTER 9 – DESIGN EXAMPLES

Why the variations?

Provides education on less common calculations. These methods may be useful when more accuracy is needed or to check computer aided designs which may employ these methods.

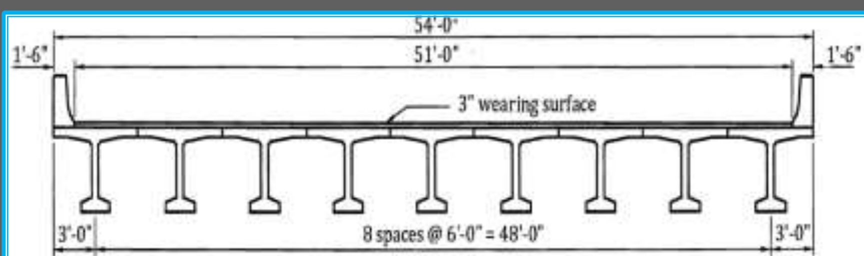
PCI EXAMPLE 9.2



Similar to Example 9.1 but have three continuous spans.

Bridge Type	Span	Cross Section	Prestress Losses	Shear
BT-72 beams with CIP composite deck	Three span continuous 110'-120'-110'	Transformed	Refined	General

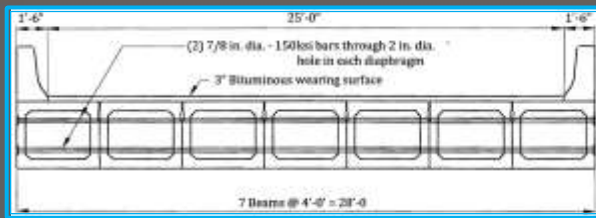
PCI EXAMPLE 9.3



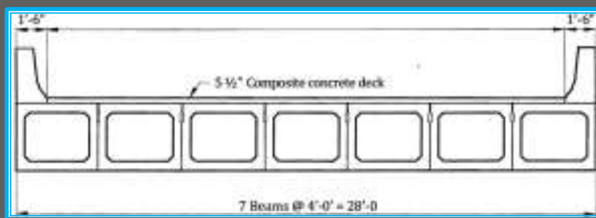
Bridge Type	Span	Cross Section	Prestress Losses	Shear
DBT-53 beams with non-composite wearing surface	One simple span 95'	Transformed	Refined	General

PCI EXAMPLE 9.4, 9.5

Example 9.4

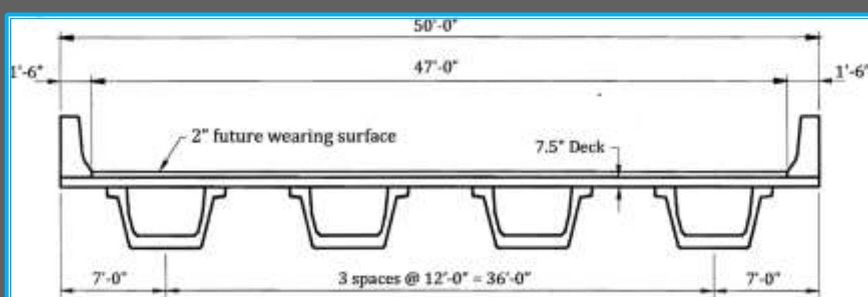


Example 9.5



	Bridge Type	Span	Cross Section	Prestress Losses	Shear
9.4	Adjacent BIII-48 beams without CIP deck	One simple span 95'	Transformed	Refined	General
9.5	Adjacent BIII-48 beams with 5.5-in. CIP deck				

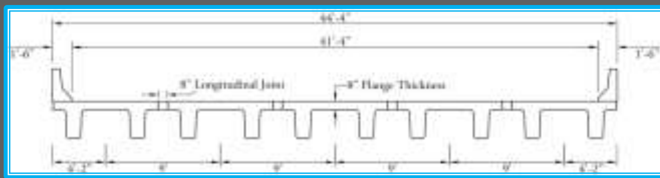
PCI EXAMPLE 9.6



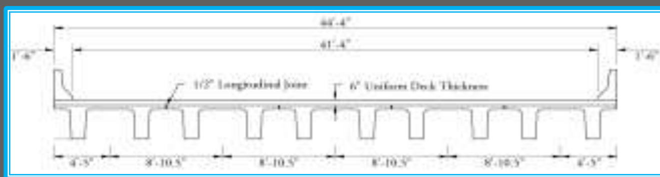
Bridge Type	Span	Cross Section	Prestress Losses	Shear
Texas U54 beams with 3-1/2-in.-thick precast panels and 4-in.-thick CIP deck	One simple span 120'	Transformed	Refined	General

PCI EXAMPLE 9.7, 9.8

Example 9.7

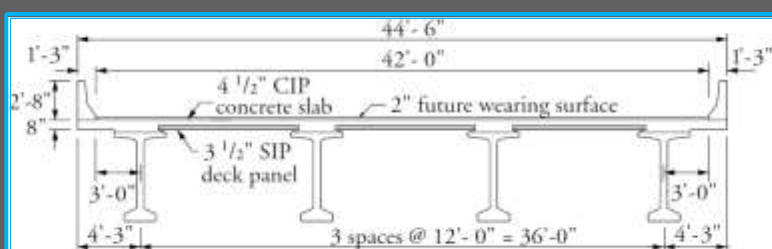


Example 9.8



	Bridge Type	Span	Cross Section	Prestress Losses	Shear
9.7	NEXT 36D Double-tee beams without CIP deck, with transverse post-tensioning	One simple span 80'	Transformed	Refined	General
9.8	NEXT 36F Double-tee beams with 6-in.-thick CIP deck and no P/T				

PCI EXAMPLE 9.10



Bridge Type	Span	Cross Section	Prestress Losses	Shear
Precast Concrete Stay-In-Place Deck Panel System	9.5 ft Panel	Transformed	Refined	General

MAJOR DIFFERENCES IN NEW MANUAL

	Old manual (2003)	New Manual (2011)
Prestress losses	Old simple method	New refined method
Shear	Old method with iterative process (moved to Appendix B5 in new AASHTO Code)	New method without the iterative process
Effective flange width	Least of: One-quarter of the effective span length; 12.0 times the average depth of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder; The average spacing of adjacent beams.	Tributary width
Fatigue	Fatigue (LF=0.75)	Fatigue I (LF=1.50) Fatigue II (LF=0.75)
Maximum reinforcement limit	With limit on maximum reinforcement	Removed in 2005

DIFFERENCE ON PRESTRESS LOSSES

(PCI Example 9.1)

	2003 Manual (Gross Section)	2011 Manual 9.1b (Gross Section)			2011 Manual 9.1a (Transformed Section)		
Prestress Losses							
		Before deck	After deck	Total	Before deck	After deck	Total
Elastic Shortening	18.60	18.60		18.60	18.90		18.90
Shrinkage	6.50	6.02	2.54	8.56	6.02	2.54	8.56
Creep	26.20	15.19	-0.96	14.23	15.45	-0.36	15.09
Relaxation	1.80	1.27	1.27	2.54	1.26	1.26	2.52
Shrinkage of deck			-1.19	-1.19		-1.19	-1.19
Total	53.10	42.73			43.87		
Stress at Transfer (Midspan)							
Top of girder	0.301	0.299			0.288		
Bottom of girder	3.266	3.273			3.337		
Stress at Service (Midspan)							
Top of girder	2.335	2.249			2.237		
Bottom of girder	-0.487	-0.034			0.154		

IMPORTANT NOTE

The AASHTO Code method of applying refined losses is to calculate the losses and gains in the prestressing steel force and then find the concrete fiber stress.

However, PCI prefers a more conservative approach.

CHAPTER 9 – DESIGN EXAMPLES 9.1a

The difference between the AASHTO *LRFD Specification* method and the method favored by PCI occurs when the gain due to deck shrinkage, Δf_{pSS} , is considered.

PCI suggests this should be found by considering deck shrinkage as a force applied to the gross composite section.

CHAPTER 9 – DESIGN EXAMPLE 9.1a

This is controversial and still under study.

Some believe the current presentation of elastic gain from deck shrinkage applied to prestress losses is unconservative because it does not correctly calculate concrete fiber stresses.

Some believe the proposed method of considering deck shrinkage as a force is too conservative; others disagree.

Some suggest using 50% of the force calculated by this method.

DIFFERENCE ON SHEAR RESISTANCE

(PCI Example 9.1)

	2003 Manual (Old general)	2011 Manual 9.1b (Appendix 5)	2011 Manual 9.1a (New general)
Method	Iterative process to calculate β and θ	Same as the old method	New method without the iterative process
Θ (deg)	23	23	29
β	2.94	2.94	4.8
V_c (Kips)	103.9	103.9	169.7
V_s (Kips)	344.6	344.6	263.9
V_c+V_s	448.5	448.5	433.6

DIFFERENCE ON EFFECTIVE FLANGE WIDTH (PCI Example 9.1)

Old manual (2003):

Effective flange width shall be the lesser of:

- (1/4) span = (120)(12/4) = 360 in.
- $12t_s$ plus greater of web thickness or 1/2 beam top flange width = (12 x 7.5) + (0.5 x 42) = 111 in.
- average spacing between beams = (9 x 12) = 108 in. (Control)

Therefore, the effective flange width is = **108 in**.

New manual (2011):

Effective flange width is taken as the tributary width perpendicular to the axis of the beam. For the interior beam, the effective flange width is calculated as one-half the distance to the adjacent beam on each side.

- 2 x (4.5 x 12) = 108 in.

Therefore, the effective flange width is = **108 in**.

DIFFERENCE ON EFFECTIVE FLANGE WIDTH (With Different Beam Spacing)

Beam Spacing	2003 Manual			2011 Manual			Difference	
	Effective flange width (in)	Area (in^2)	I (in^4)	Effective flange width (in)	Area (in^2)	I (in^4)	Area	I
8ft	96	1348	1.07E+06	96	1348	1.07E+06	0.0%	0.0%
9ft	108	1419	1.10E+06	108	1419	1.10E+06	0.0%	0.0%
10ft	111	1437	1.11E+06	120	1490	1.13E+06	3.7%	1.8%
11ft	111	1437	1.11E+06	132	1560	1.16E+06	8.6%	4.5%
12ft	111	1437	1.11E+06	144	1630	1.19E+06	13.4%	6.8%

DIFFERENCE ON FATIGUE

2003 Manual	2011 Manual
Fatigue LF = 0.75	Fatigue I: LF = 1.50 Fatigue II: LF = 0.75
In regions of compressive stress due to permanent loads and prestress in reinforced and partially prestressed concrete components, fatigue shall be considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the Fatigue load combination as specified in Table 3.4.1-1in combination with the provisions of Article 3.6.1.4.	In regions of compressive stress due to permanent loads and prestress in reinforced and partially prestressed concrete components, fatigue shall be considered only if this compressive stress is less than the maximum tensile live load stress resulting from the Fatigue I load combination as specified in Table 3.4.1-1in combination with the provisions of Article 3.6.1.4.
No specific requirement for fully prestressed components	For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed 0.40f 'c after losses.

Fatigue

When checking the Service I load combination, the stress at the top of the girder due to permanent loads was found to be:

f_t = +1.737 ksi

From the table shown previously, the moment at midspan due to the fatigue truck is:

M_f = 776.9 k-ft.

Fatigue
(using example 9.1a)

$$f_{tgf} = \frac{1.5M_f}{S_{ttc}} = \frac{1.5(776.9k-ft)(12)}{64161in^3} = +0.218ksi$$

$$f_{tgf} + 0.5f_{tg} = 0.218 + 0.5(1.737) = 1.087ksi$$

$$< 0.4f_c' = 0.4(6.5) = 2.8ksi$$

This condition should be checked at all sections of the girder.

CHAPTER 9 – DESIGN EXAMPLES
Summary

- 11 Design Examples
- Various cross sections included
- Adjacent and stringer bridges.
- Simple span and continuous bridges
- Gross and transformed properties.
- Refined and simplified losses
- Sectional model and simplified model for shear.

CHAPTER 18 – Load Rating

18.1 – Overview of Load Rating
18.2 – Loads and Distributions
18.3 – Rating Methodology
18.4 – Rating by Load Testing
18.5 – Load Rating Report
18.6 – Rating Example
18.7 - References

CHAPTER 18 – Load Rating

18.1 Overview of Bridge Load Rating
18.2 Loads and Distribution
18.3 Rating Methodology
18.4 Rating by Load Testing
18.5 Load Rating Report
18.6 Rating Example
18.7 References

CHAPTER 18 – Load Rating

- Completely rewritten
- ASD, LFD and LRFR information is now compliant with the *AASHTO Manual for Evaluation of Bridges .*
  - Replaces *AASHTO Manual for Condition Evaluation of Bridges*

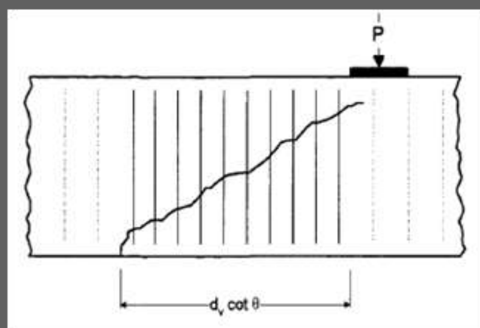
CHAPTER 18 – Load Rating

This Chapter provides the basic definitions for rating:

Inventory Rating — The load that can safely utilize the bridge for an indefinite period of time. Generally this analysis is performed in accordance with the design specifications.
Operating Rating — The absolute maximum permissible load to which the bridge can be subjected. This analysis may utilize posting avoidance techniques as specified by the jurisdiction.
Load Rating — The process of determining the live load capacity of a bridge based on its current conditions through either analysis or load testing.
Rating Factor — The ratio of available live load moment or shear capacity to the moment or shear produced by the load being investigated.
Routing Vehicle — A state defined permit truck that is used to create overload maps for using in prescribing which arterial maybe be used by a set fleet of Specialize Hauling Vehicles (SHV).

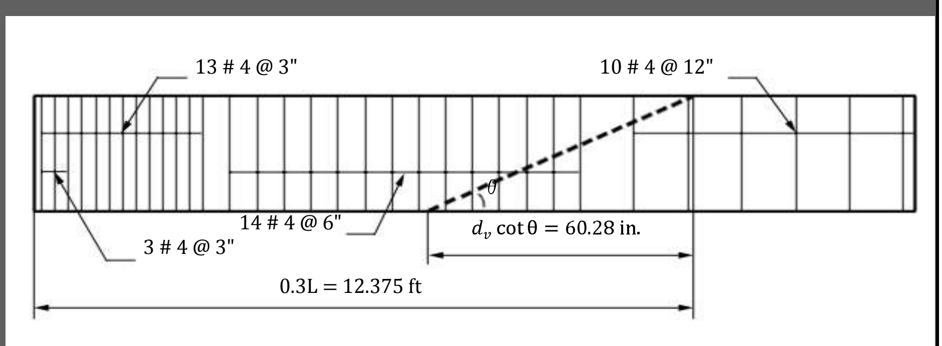
CHAPTER 18 – Load Rating

This chapter also covers the exact method of determining shear resistance by properly counting all the stirrups which cross the failure plane.



CHAPTER 18 – Load Rating

Here is a illustration of how the exact method is applied:



CHAPTER 18 – Load Rating

The difference is illustrated here:

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v = 0.0316(2.399)\sqrt{8.5}(6)(40.6) = 52.07\text{kips}$$

Code:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.2(60)(40.6)\text{Cot }33.67)}{12} = 60.28 \text{ kips}$$

Exact:

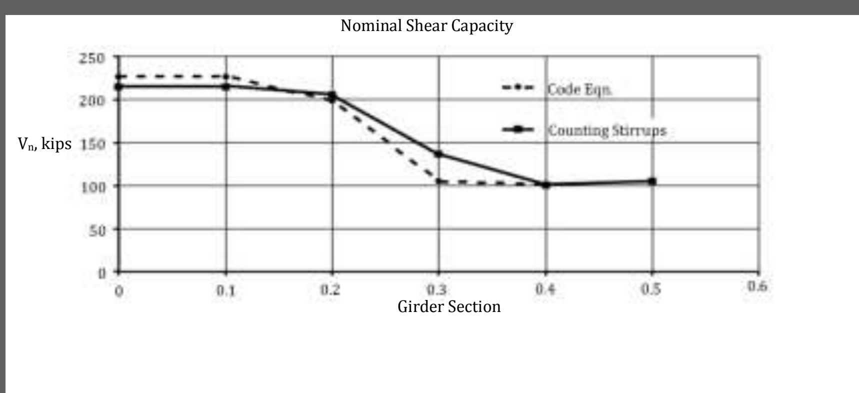
$$V_s = 8\text{stirrups}(0.2\text{in}^2/\text{stirrup})(60\text{ksi}) = 96\text{kips}$$

$$V_{n,code} = 112.4\text{kips}$$

$$V_{n,exact} = 148.1\text{kips}$$

$$\Delta = 32\%$$

CHAPTER 18 – Load Rating



CHAPTER 18 – Load Rating

18.6.10 Summary of Ratings

In summary, looking at the older structure that was not designed with the new reliability based *LRFD Specifications*, one arrives at the following conclusions:

Standard Specifications Rating Factors

	Inventory Rating (Notional load)	Operating Rating
LFD Strength (HS20)	1.25	2.09 (HS41.4)
LFD Service (HS20)	1.21	
LFD Proof Test (HS20)	2.50	4.32 for interior use (HS33)

LRFD Specifications Rating Factors

	Inventory Rating	Operating Rating
LRFD Strength I (HL-93)	1.18	1.53
LRFD Service III (HL-93)	1.15	
LRFD Service I (HL-93)		2.06
LRFD Strength II (HL-93) Routine Blanket Permit in mixed traffic		1.00
LRFD Service I(HL-93) Routine Blanket Permit in mixed traffic		1.58
LRFD Strength II(FL-120) Escorted single trip without others lanes loaded		2.29
LRFD Strength II(FL-120) Escorted single trip with other lanes loaded		1.17 (HS39.1)

CHAPTER 19- REPAIR AND REHABILITATION

CHAPTER 20- Piles

CHAPTER 21- Recreational Bridges

Issues in Next Release (1Q2012)

SUMMARY

- The PCI Bridge Design Manual has been completely updated through the AASHTO LRFD Specifications, 5th Edition with 2011 interim.
- The update includes the 2011 versions of other applicable specifications.
- Design Examples of Chapter 9 have been expanded to include more bridge types and to illustrate different design methods.

The new Bridge Design Manual is the perfect reference book for concrete bridge design.

It is also an excellent educational tool!

Thank you for your attention.

Questions?

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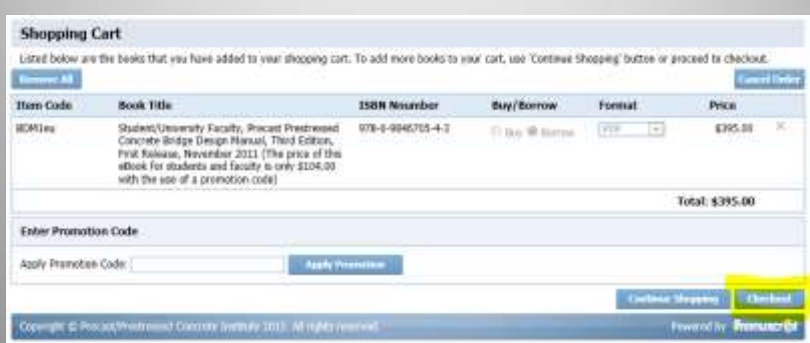

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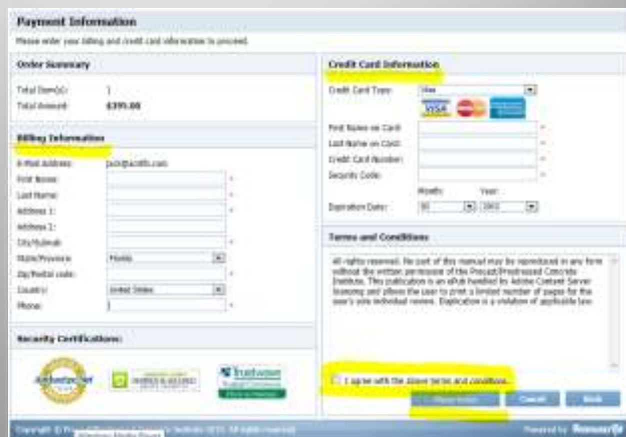

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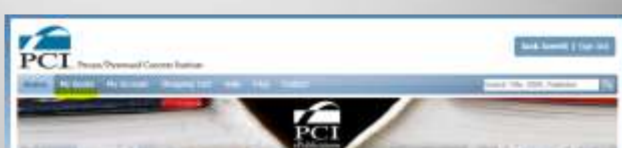

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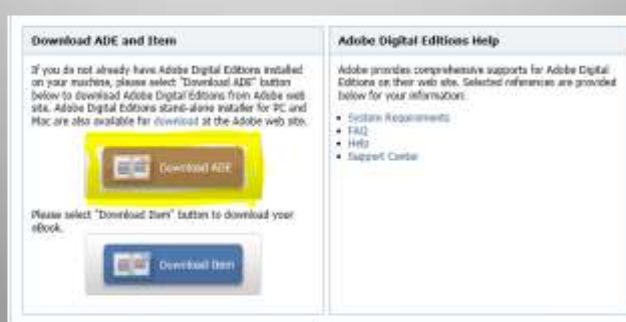

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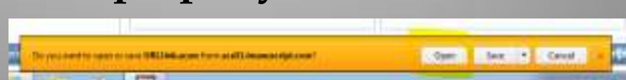

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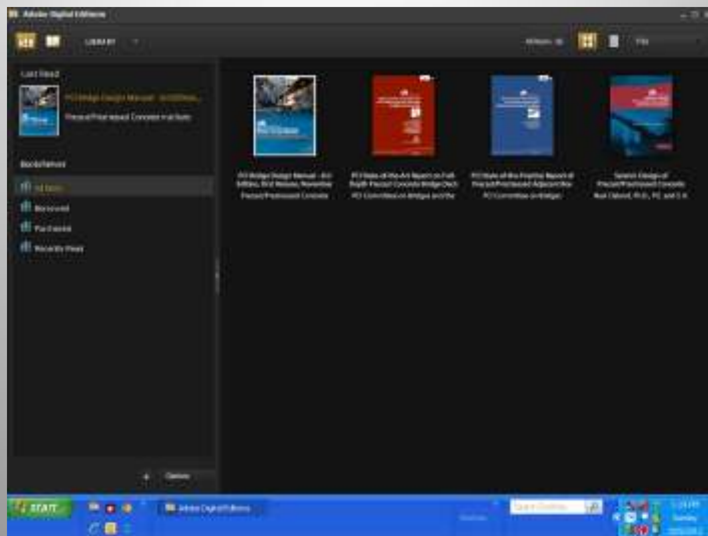
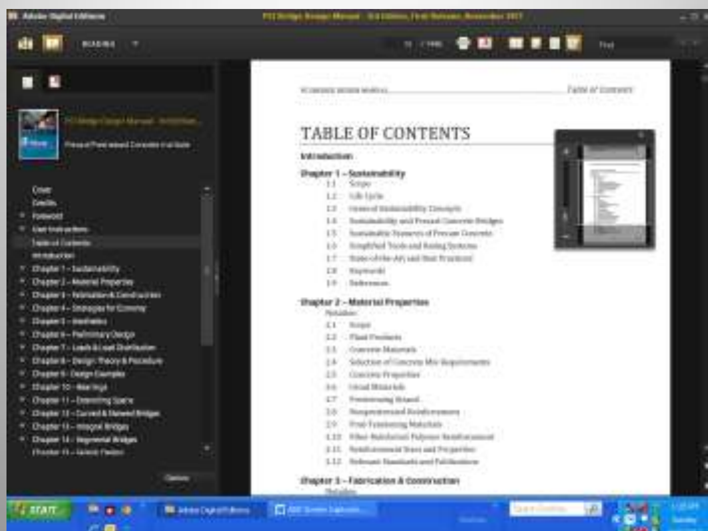


TOC and Bookmarks



Navigating

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Detailed Calculations and Links

- MathCad like Calculation with full detail
- Referenced
  - AASHTO
  - Research
  - NCHRP

Thank you for your attention.

Questions?