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Course Description

Session L2: Plate Girder Design and Stability

October 19, 2017

This session will focus on the design and behavior of plate girders in steel bridges. Methods of preliminary sizing of the girders will be discussed along with an overview of the strength evaluation of composite girders. The AASHTO strength provisions for the evaluation of member and system stability will be covered. Member and system stability topics will include the evaluation of local buckling, lateral torsional buckling, as well as system buckling of narrow steel bridge systems. The principles will be supported through design examples.

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Learning Objectives

- Gain an understanding of the design and behavior of plate girders in steel bridges.
- Gain an understanding of preliminary sizing of girders and strength evaluation of composite girders.
- Become familiar with AASHTO strength provision evaluation of member and system stability.
- Become familiar with stability topics including local buckling, lateral torsional buckling and system buckling of narrow steel bridge systems.

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Introduction to Steel Bridge Design

Session L2: Plate Girder Design and Stability

Presented by
Todd Helwig
University of Texas
Austin, Texas

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Introduction to Steel Bridge Design

- R1: Introduction to Bridge Engineering
- R2: Introduction and History of AASHTO LRFD Bridge Design Specifications
- R3: Steel Material Properties
- R4: Loads and Analysis

- L1: Steel Bridge Fabrication
- **L2: Plate Girder Design and Stability**
- L3: Effects of Curvature and Skew
- L4: Fatigue and Fracture Design

Session Outline

- Overview of Local and Global Stability Limit States
- Composite Girder Design
- Girder Design Preliminary/Final
  - Finished Bridge (Composite Girder)
  - Construction Stage (Non-Composite Girder)
- Bracing Layout and System mode of Buckling

Responsibilities of Designer

- Design and behavior of the girders during placement of the concrete bridge deck (construction stage – steel girder generally carries the full construction load)
- Design and behavior of the girders in the finished bridge (composite section).
- The design requirements and corresponding limit states of the girders during these two stages are very different

Review of Basic Stability Requirements of Steel Bridge Girders

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Material Properties of Steel

- Structural steel is an extremely efficient material in terms of both strength and stiffness.
- Comparing typical steel and concrete properties:

Material	Strength	Stiffness (E)
Steel	Yield: 36~100 ksi	29000 ksi
Concrete	f'_c = 3~7 ksi	3000~5000 ksi

- While concrete is primarily only relied upon in compression, steel is essentially equally strong in both tension and compression (from a material perspective).

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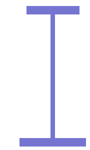
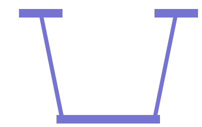
Cross Sectional Proportions

- Because steel has a high strength and stiffness, efficient cross sections will often consist of relatively slender plate elements
- The resulting cross sections are often controlled by stability limit states (particularly during construction)
- The stability limit states include both local and global instabilities (buckling)





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What is Buckling?

- Buckling is a compression driven phenomenon in which the element experiences a loss of stiffness.
- In most situations, the loss of stiffness is a gradual effect; however, in cases such as a bracing failure, buckling can be a sudden catastrophic failure.
- As the buckling load is approached small changes in the applied loading can result in large deformations.

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Loss of Stiffness Due to Buckling

Video courtesy of Ron Ziemian – Bucknell University

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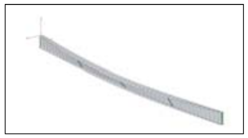
Stability Limit States

- Buckling Modes:
  - Local (plate) Buckling
    - Flanges (flange local buckling)
    - Web (web bend buckling)
  - Member buckling
    - Lateral-Torsional Buckling, Geometry, unbraced length, support conditions
  - Global/System Buckling
    - Buckling of Multiple Girders Acting as A Unit/System



Local Buckling versus Global Buckling

- Local buckling vs. global buckling can have very different impacts on the behavior
- Post-buckling behavior
  – Member ultimate strength ~ member/global buckling capacity
  – Substantial post-buckling strength in plates
  – Plate girder web shear capacity >> buckling prediction

Plate Buckling

Elastic plate buckling stress:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)\left({}^{b}/_{t}\right)^2}$$

k = plate buckling coefficient

E = modulus of elasticity of plate material (ksi)

μ = Poisson's ratio = 0.3

b = plate width (in)

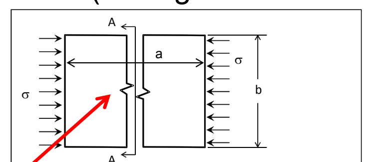
t = plate thickness (in)

Plate Buckling

3 Types of plate buckling coefficient k:

- Axial compression (see figure for example)
- Bending
- Shear

"Long" plate in axial compression, so a/b >> 1



Case	Edge Support	k	Buckled Shape Along Section A-A
1	Both Edges SS	4.00	b
2	One Edge SS, the other fixed	5.42	
3	Both Edges Fixed	6.97	
4	One Edge SS, Other Free	0.42	
5	One Fixed, Other Free	1.28	

SS – Simply Supported

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Plate Buckling

Local buckling in steel beam/girder applications can exist in several different limit states:

- Flange Local Buckling
- Web Local Buckling
- Web Shear Buckling
- Web Crippling or other effects around concentrated forces

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Flange Local Buckling (FLB)

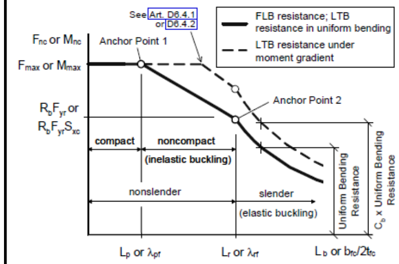

Figure C6.10.8.2.1-1—Basic Form of All I-section Compression-Flange Flexural Resistance Equations

Flange Slenderness:

$$\lambda = \frac{b_f}{2t_f}$$

Compact Slenderness limit:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}}$$ (λ_{pf} = 9.2, F_y = 50 ksi)

Non-Compact Slenderness limit:

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_{yr}}}$$ (λ_{rf} = 16.1, F_y = 50 ksi)

Note: Yield stress including residual effects $F_{yr} = 0.7F_y$

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Web Local Buckling

Web Slenderness Compact and Noncompact Limits (AASHTO Appendix A6.2.1):

$$\frac{2D_{cp}}{t_w} < \lambda_{pw(D_{cp})}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}}$$ λ_{rw} = 137 for Gr. 50

Most webs are designed with λ close to λ_{rw}

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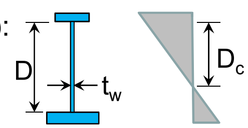
Web Bend Buckling

Web Load Shedding Factor: AASHTO 6.10.1.10.2 (for Slender web members):

$$\frac{2D_c}{t_w} > \lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}}$$



The LTB buckling expressions use an R_b factor that accounts for the impact of local web buckling on the global LTB resistance.

$$R_b = 1 - \left(\frac{a_{wc}}{1200 + 300a_{wc}}\right)\left(\frac{2D_c}{t_w} - \lambda_{rw}\right) \le 1.0$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}}$$ b_{fc}, t_{fc} = comp. flange width and thickness

R_b = 1.0 for $2D_c/t_w < \lambda_{rw}$ (For compact and non-compact webs)

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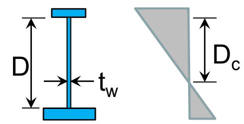
Web Bend Buckling (Unstiffened Webs)

Web Bend Buckling: AASHTO 6.10.1.9
(for Slender web members):

$$\frac{2D_c}{t_w} > \lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}}$$



Webs without Longitudinal Stiffeners – Nominal web bend buckling resistance:

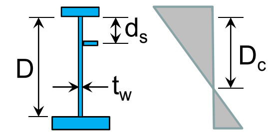
$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad k = \frac{9}{\left(\frac{D_c}{D}\right)^2}$$

D = web depth, D_c = depth of web in compression.

Note – compare to previous plate buckling expression $\frac{\pi^2}{\left(12\left(1-\mu^2\right)\right)} = 0.9$ (μ = 0.3)

Longitudinal Stiffener – Web Buckling

Longitudinal Stiffener is req'd for $D/t_w > 150$



$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2}$$

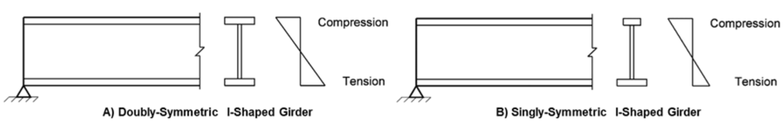
Webs with Longitudinal Stiffeners – use the following plate buckling coefficient in the buckling expression:

If $\frac{d_s}{D_c} \geq 0.4$ $\qquad k = \frac{5.17}{\left(\frac{d_s}{D}\right)^2} \geq \frac{9}{\left(\frac{D_c}{D}\right)^2}$

If $\frac{d_s}{D_c} < 0.4$ $\qquad k = \frac{11.64}{\left(\frac{(D_c - d_s)}{D}\right)^2}$

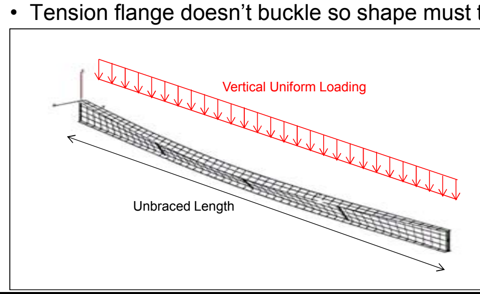
Lateral Torsional Buckling

- Bending produces both tensile and compressive stresses
  - Doubly-symmetric shape
    - Neutral axis at mid-depth
    - Maximum tensile and compressive stresses equal
  - Singly-symmetric shape
    - Neutral axis closer to larger flange
    - Maximum bending stress at smaller flange



Lateral Torsional Buckling

- Girder buckling mode: Lateral-Torsional
  - Tensile & compressive stresses produced by bending
  - Result: lateral translation and twisting of cross-section
    - Compression flange buckles & laterally translates
    - Tension flange doesn't buckle so shape must twist



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Torsional Resistance

- 2 types of torsional resistance
  - Saint-Venant torsional stiffness (sometimes referred to as the uniform resistance). Related to GJ
  - Warping torsional stiffness (sometimes called the non-uniform torsional resistance)
- Shape-dependent
- Restraint-dependent

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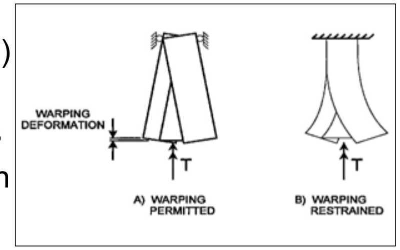
Torsional Buckling (con.)

- Warping torsion is non-uniform (dependent on L)
- Bending deformation in plane of individual plates
- Figure A: Flanges remain straight; only St.-Venant stiffness
- Figure B: Warping stiffness = resistance to warping deformation



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LTB – Doubly Symmetric Shape

- Timoshenko's equation for elastic critical buckling moment M_{cr} (k-in):

$$M_{cr} = \frac{\pi}{L_b}\sqrt{EI_yGJ + \frac{\pi^2E^2I_yC_w}{L_b^2}}$$

  - L_b = unbraced length (in) = spacing between cross-frames
  - E = modulus of elasticity (ksi)
  - G = shear modulus (ksi)
  - J = torsional constant (in^4)
  - C_w = warping constant (in^6)
  - I_y = weak-axis moment of inertia (in^4)

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AASHTO LTB Provisions

- AASHTO Appendix A6 equation A6.3.3-8 for elastic LTB
  – Applicable for non-compact and compact (doubly- and singly – symmetric) sections:

$$F_{cr} = \frac{C_b\pi^2E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h}\left(\frac{L_b}{r_t}\right)^2}$$

Where:
- C_b = moment modification factor (later)
- S_{xc} = elastic section modulus to comp. flange (in^3)
- r_t = effective lateral-torsional radius of gyration (in)
- h = depth between centerline of flanges (in)

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AASHTO LTB

- AASHTO Appendix A6 equation for effective radius of gyration r_t (in), used in previous slide:

$$r_t = \frac{b_{fc}}{\sqrt{12\left[1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right]}}$$

- Where:
  - b_{fc} = width of compression flange (in)
  - t_{fc} = thickness of compression flange (in)
  - D_c = depth of web in compression (in)
  - t_w = thickness of web (in)

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AASHTO Appendix A6 LTB

- AASHTO equation for lateral torsional buckling:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h}\left(\frac{L_b}{r_t}\right)^2}$$

  - 1st term under radical: Warping torsional stiffness
  - 2nd term under radical: Saint-Venant torsional stiffness
  - Applicable to doubly-symmetric and singly-symmetric shapes
- Timoshenko equation vs. AASHTO equation
  - For doubly-symmetric I-section, ~ identical solutions

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AASHTO Section 6.10.8.2.3

- AASHTO equation for lateral torsional buckling:

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

- The AASHTO main body equation is intended for slender web sections – which is why the web load shedding factor R_b is included.
- Many engineers use this equation even for compact and non-compact sections.

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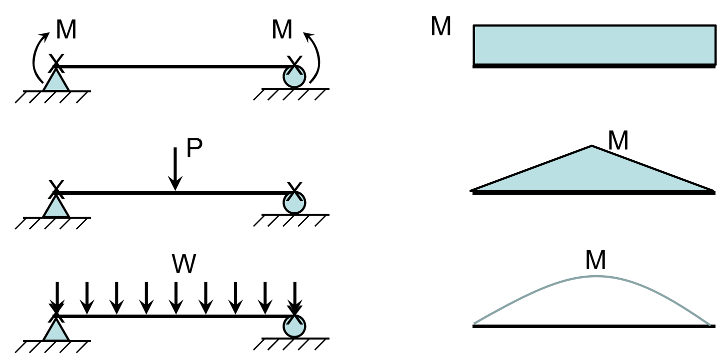
Effects of Moment Gradient



X – Brace

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Moment Gradient Effect (AASHTO)

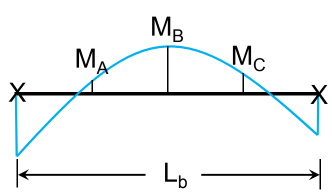
- AASHTO LRFD moment modification factor
  - $C_b = 1.75 - 1.05 f_1/f_2 + 0.3(f_1/f_2)^2 \leq 2.3$, where
    - f_2 = largest compressive bending stress at one end of unbraced length (ksi); taken as 0 if stress is zero / tensile at both ends
    - f_0 = compressive bending stress at opposite end of unbraced length (ksi); negative if in tension
    - f_{mid} = compressive bending stress at middle of unbraced length (ksi)
    - f_1 = f_0 if variation of moment is concave between brace points
    - f_1 = $2f_{mid} - f_2 \geq f_0$ otherwise (i.e., linear extrapolation)

Moment Gradient Effect (AASHTO)

- AASHTO LRFD moment modification factor, continued
  - $C_b = 1$ for unbraced cantilevers and members where $f_{mid}/f_2 > 1$ or $f_2 = 0$, where
    - f_2 = largest compressive bending stress at one end of unbraced length (ksi); taken as 0 if stress is zero / tensile at both ends
    - f_{mid} = compressive bending stress at middle of unbraced length (ksi)

Moment Gradient Effect (AISC)

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C}$$



X – Brace

M_{max} can occur anywhere along the unbraced length

Absolute value of moments are used in equation.

Inelastic LTB

$L_b < L_p$ – Yielding Limit State

$L_p < L_b < L_r$ – Inelastic LTB

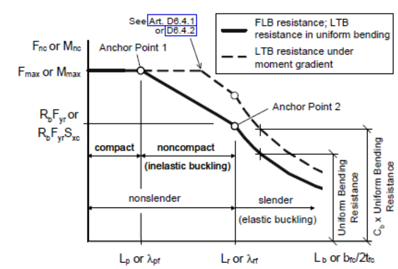


Figure C6.10.8.2.1-1—Basic Form of All I-section Compression-Flange Flexural Resistance Equations

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Review of Composite Girder Fundamentals

Composite Girder Design

We are going to primarily focus on the behavior in the positive moment region. Subsequently, we will provide a brief discussion on the negative moment region.

Composite Girder Strength

SECTION 6: STEEL STRUCTURES

APPENDIX D6—FUNDAMENTAL CAL(

CALCULATIONS FOR FLEXURAL MEMBERS

D6.1—PLASTIC MOMENT

The plastic moment, M_p, shall be calculated as the moment of the plastic forces about the plastic neutral axis. Plastic forces in steel portions of a cross-section shall be calculated using the yield strengths of the flanges, the web, and reinforcing steel, as appropriate. Plastic forces in concrete portions of the cross-section that are in compression may be based on a rectangular stress block with the magnitude of the compressive stress equal to $0.85f'_c$. Concrete in tension shall be neglected.

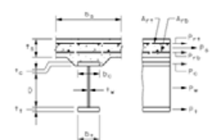
The position of the plastic neutral axis shall be determined by the equilibrium condition that there is no net axial force.

Composite Girder Strength

6-314 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table D6.1-1—Calculation of $\bar{Y}$ and M_p for Sections in Positive Flexure

Case	PNA	Condition	$\bar{Y}$ and M_p
I	In Web	$P_t + P_w \ge P_c + P_s + P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + (D-\bar{Y})^2\right] + [P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t]$
II	In Top Flange	$P_t + P_w + P_c \ge P_s + P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$ $M_p = \frac{P_c}{2t_c}\left[\bar{Y}^2 + (t_c - \bar{Y})^2\right] + [P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t]$
III	Concrete Deck, Below P_{rb}	$P_t + P_w + P_c \ge \left(\frac{c_{rb}}{t_s}\right) P_s + P_{rb} + P_{rt}$	$\bar{Y} = (t_s)\left[\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right]$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + [P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t]$
IV	Concrete Deck, at P_{rb}	$P_t + P_w + P_c + P_{rb} \ge \left(\frac{c_{rb}}{t_s}\right) P_s + P_{rt}$	$\bar{Y} = c_{rb}$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + [P_{rt} d_{rt} + P_c d_c + P_w d_w + P_t d_t]$
V	Concrete Deck, Above P_{rb} Below P_{rt}	$P_t + P_w + P_c + P_{rb} \ge \left(\frac{c_{rt}}{t_s}\right) P_s + P_{rt}$	$\bar{Y} = (t_s)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + [P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t]$
VI	Concrete Deck, at P_{rt}	$P_t + P_w + P_c + P_{rb} + P_{rt} \ge \left(\frac{c_{rt}}{t_s}\right) P_s$	$\bar{Y} = c_{rt}$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + [P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t]$
VII	Concrete Deck, Above P_{rt}	$P_t + P_w + P_c + P_{rb} + P_{rt} < \left(\frac{c_{rt}}{t_s}\right) P_s$	$\bar{Y} = (t_s)\left[\frac{P_{rb} + P_c + P_w + P_t + P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + [P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t]$



in which:

$P_{rt} = F_{yrt} A_{rt}$
$P_s = 0.85 f'_c b_s t_s$
$P_{rb} = F_{yrb} A_{rb}$
$P_c = F_{yc} b_c t_c$
$P_w = F_{yw} D t_w$
$P_t = F_{yt} b_t t_t$

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Composite Girder Strength (positive bending)

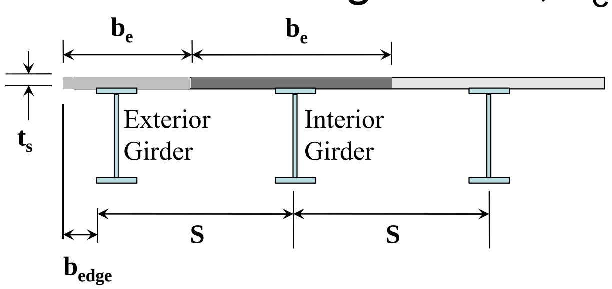
The composite girder strength is based upon an ultimate strength model that utilizes the following basic assumptions:

- Concrete above the plastic neutral axis is at $0.85f'_c$ in compression and concrete below the neutral axis does not contribute
- Steel above the neutral axis is at F_y in compression
- Steel below the neutral axis is at F_y in tension

Steps to Determining Composite Girder Strength (positive bending)

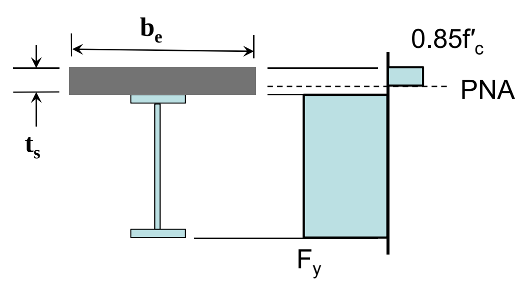
- Identify effective deck width, b_e, for composite girder
- Locate Plastic Neutral Axis
  - May be in concrete deck
  - May be in steel girder flange
  - May be in steel girder web
- Draw free body from stress diagram and sum moments about any desired location to find ultimate strength.

Effective Flange Width, b_e



AASHTO Section 4.6.2.6

Locate the Plastic Neutral Axis (PNA)



The PNA is found from equilibrium -
Compressive Resultant = Tensile Resultant

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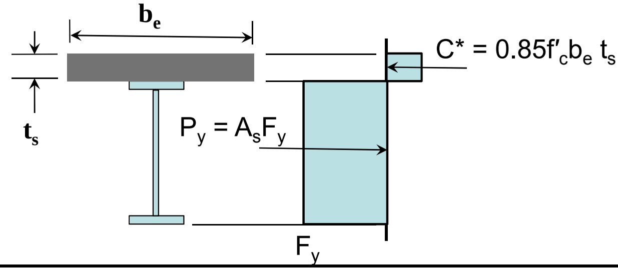
General Location of PNA

The general location of the PNA can be found by comparing three idealized resultants found from equilibrium:

Compressive Resultants = Tensile Resultants



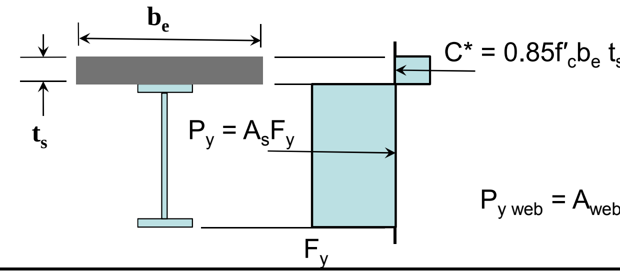
General Location of PNA

PNA in Slab if $C^* > P_y$

PNA in Steel flange if $P_y > C^* > P_{y\ web}$

PNA in Steel web if $P_{y\ web} > C^*$

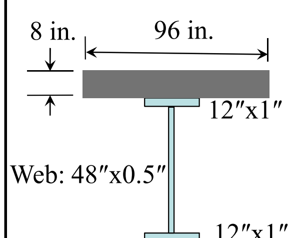
Doubly-symmetric Steel Girder



$P_{y\ web} = A_{web}F_y$

Example 1 – PNA in Concrete Deck

Consider the doubly-symmetric steel (F_y=50 ksi) girder (A_s = 48 in^2) with a concrete deck with f'_c=4ksi. Find M_p of the composite girder.



$C^* = 0.85f'_c b_e\, t_s$

C^* = 0.85(4 ksi) x(96") x (8") =2610 k

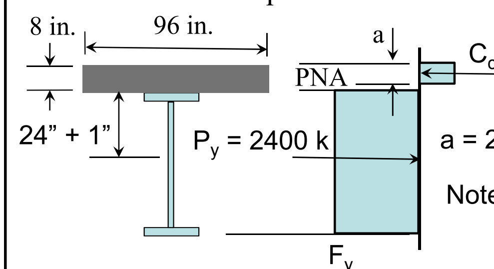
$P_y = A_sF_y$ = 48 in^2 x 50 ksi = 2400 k

C^* =2610 k > 2400 k = P_y

PNA is in Concrete Deck

Example 1 (Continued)

Equilibrium: C = T = 2400 k



$C_c = 0.85f'_c b_e\, a$ = 2400 k

a = 2400/(0.85x4x96) = 7.35"

Note: a < 8" = t_s as expected

M_p = 2400 k (24" + 1" + 8" – 7.35"/2) = 70,400 k-in = 5865 k-ft.

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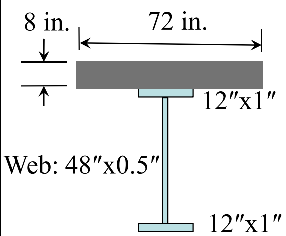
Example 2 – PNA in Steel Section

Consider the doubly-symmetric steel (F_y=50 ksi) girder (A_s = 48 in^2) with a concrete deck with f'_c=4ksi. Find M_p of the composite girder. Girder Spacing = 72 in.



$C^* = 0.85f'_c b_e t_s$

$C^* = 0.85(4\text{ ksi}) \times (72") \times (8") = 1960\text{ k}$

$P_y = A_s F_y = 48\text{ in}^2 \times 50\text{ ksi} = 2400\text{ k}$

$P_{y\,web} = A_{web} F_y = 24\text{ in}^2 \times 50\text{ ksi} = 1200\text{ k}$

$P_y = 2400\text{ k} > 1960\text{ k} = C^*$

$C^* = 1960\text{ k} > 1200\text{ k} = P_{y\,web}$

PNA is in Steel Flange

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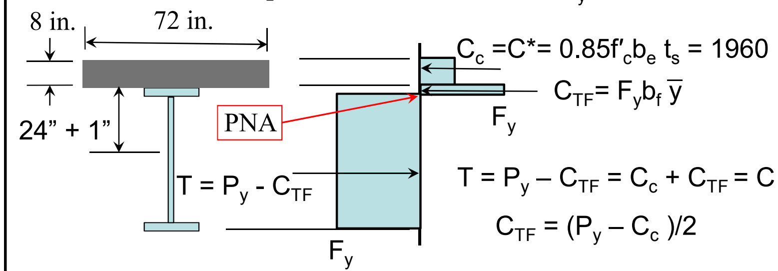
Example 2 (Continued)

Equilibrium: C = T $\quad P_y$ = 2400 k



$$\bar{y} = \frac{(P_y - C_c)}{(2b_f F_y)} = \frac{(2400k - 1960k)}{2x12"x50ksi} = \underline{\underline{0.37\ in.}}$$

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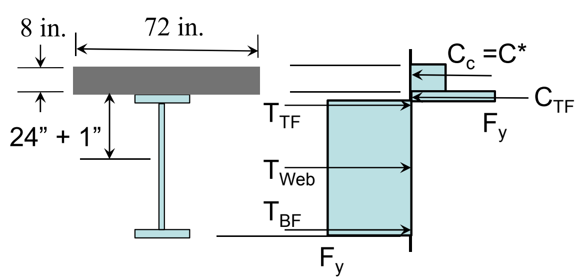
Example 2 (Continued)

In Example 1, because the entire steel girder was in tension – we knew where the force resultant of the stresses in the steel section was positioned. In this problem, there are several force resultants.



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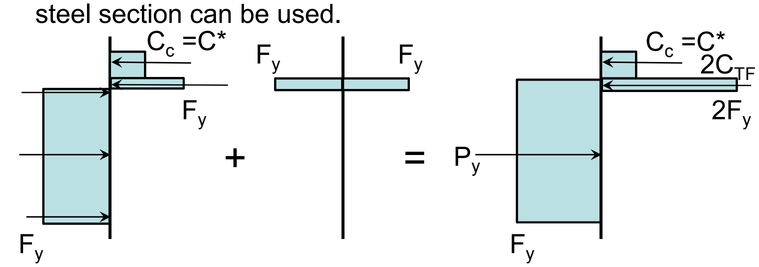
Example 2 (Continued)

When the PNA is in the top flange, a much more direct solution is possible using a "Modified Stress Diagram". The benefit of this diagram is that a single force resultant in the steel section can be used.



We can sum moments about the resultant in the compression flange of the steel section ie. $\bar{y}/2$ from top of steel section.

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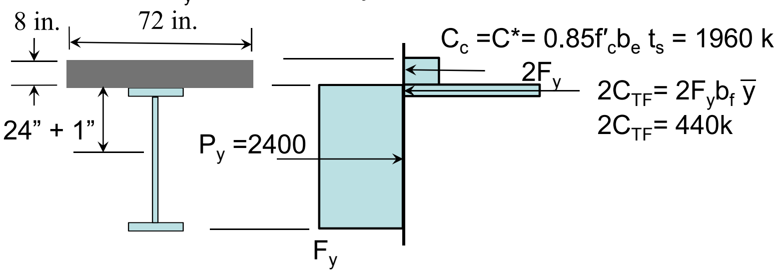
Example 2 (Continued)

P_y = 2400 k $\overline{y}$ = 0.37 in.



M_p = 2400 k (24" + 1" – 0.37"/2) + 1960(8"/2 + (0.37"/2)

M_p = 67,760 k-in = 5645 k-ft.

Shear Stud Strength

- Section 6.10.10.4.3

$$Q_n = 0.5A_{SC}\sqrt{f_c' E_c} \le A_{SC}F_u$$

Q_n = Shear strength of a single connector (kips)

A_{SC} = (π/4)(d_s)^2 = Area of shank of connector (in^2)

f'_c = 28-day compressive strength of concrete (ksi)

E_C = modulus of elasticity of concrete (ksi)

F_u = ultimate tensile strength of stud material (ksi)

Strength of Shear Stud

Consider ¾ in diameter shear studs with F_u=60 ksi:

$$Q_n = 0.5A_{SC}\sqrt{f_c' E_c} \le A_{SC}F_u$$

f'_c (ksi)	Q_n (kips)
3.0	20.9
3.5	23.5
4.0	26.0

Shear Stud Design

- The number of shear studs that are required is based upon an ultimate strength approach.
- A uniform spacing is used between the shear studs instead of spacing them based upon elastic shear stresses.
- Enough shear studs should be provided between the point of zero bending and the maximum positive moment.

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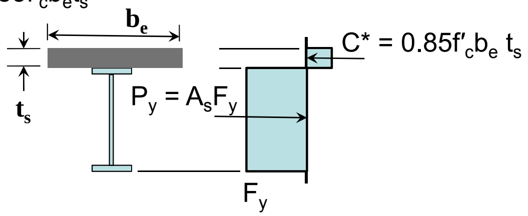
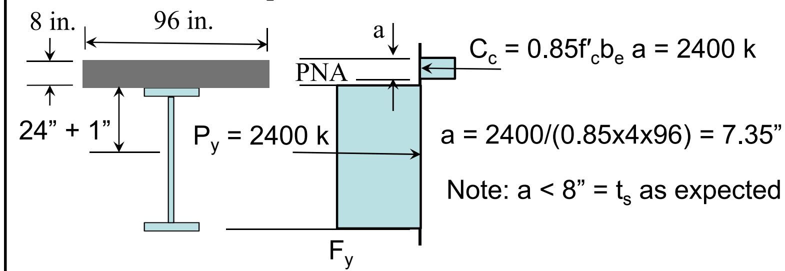
Shear Stud Design

- The number of shear studs that are required is based upon the compressive resultant in the concrete deck.
- Therefore, the shear that needs to be developed is the <u>smaller</u> of either:
  1) $P_y = A_s F_y$
  2) $C^* = 0.85f'_c b_e t_s$



Revisit Example 1

Equilibrium: C = T = 2400 k

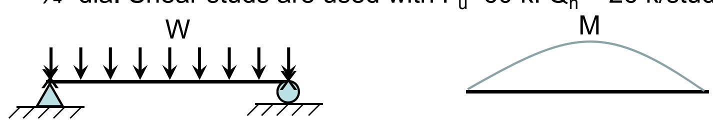


M_p = 2400 k (24” + 1” + 8” – 7.35”/2) = 70,400 k-in = 5865 k-ft.

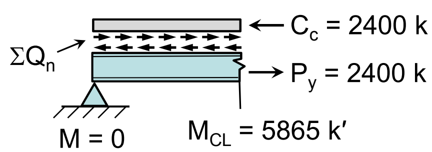
Recall that f'_c=4 ksi

Revisit Example 1

For this example, assume the beam is simply supported. ¾” dia. Shear studs are used with F_u=60 k: Q_n = 26 k/stud



Consider half the beam:



Studs/half beam = 2400k/26k/stud = <u>92 studs</u> <u>(184 total studs)</u>

Follow AASHTO Guidelines on stud spacing and also check for fatigue

Composite Strength in Negative Flexure

- While it is possible to design for composite action in the negative moment region, this typically has very limited benefit and therefore is usually not considered.
- Instead the steel girder is usually proportioned to resist the entire moment in the negative moment region. This usually results in a doubly symmetric shape in the negative moment region.

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Composite Strength in Negative Flexure

Although shear studs are not "required" in the negative moment region, it is usually a good idea to provide some connectors to better control cracking in the concrete deck.

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Preliminary Girder Sizing

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Preliminary Sizing

- A major difference between building and bridge design is the predominant use of rolled sections versus built-up sections.
- Buildings predominantly make use of rolled sections and beam selection is facilitated by strength design tables/charts.
- Bridges predominantly make use of built up sections that require the designer to initiate the design with preliminary sizes and iterate based upon the strength requirements in the finished bridge and the demand during construction.

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Typical Span/Depth Ratios

- A primary starting point in preliminary sizing is often based upon typical span/depth ratios.
- Table 2.5.2.6.3-1 in AASHTO provides some general guidelines on target L/d ratios for the steel beams that can be used to begin the sizing:
  - Simple spans: L/d < 25~30
  - Continuous spans: L/d < 30~35

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Deck Forming Systems

- Many steel bridges are constructed using stay-in-place (SIP) forming systems.
- These forms provide a working surface relatively quickly for the construction personnel and as the name implies, the forms stay on the bridge permanently.
- The forms are supported on a cold-formed angle that allows the contractor to account for variations in the flange thickness along the girder length and differential camber across the bridge width.

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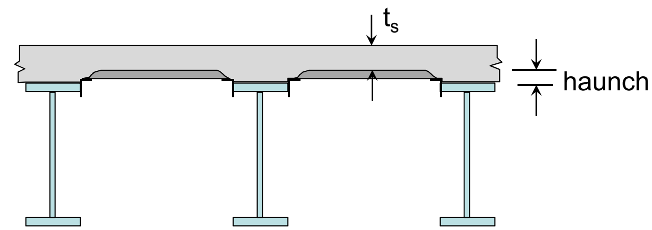
Deck Forming Systems



The thickness of the slab for the concrete deck is typically taken from the top of the formwork to the top of the slab.

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Stay-in-Place Forms

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Preliminary Sizing

- The sizing often begins with a preliminary analysis to obtain a design moment, M_u, considering dead and live loading (HL93 Truck and Lane Loading).
- Additional fundamental decisions prior to sizing include girder spacing, grade of steel, 28-day compressive strength of concrete.
- We can begin by assuming that the PNA will be in the concrete deck and rearrange the expression we had for M_p to solve for the required area of the steel.

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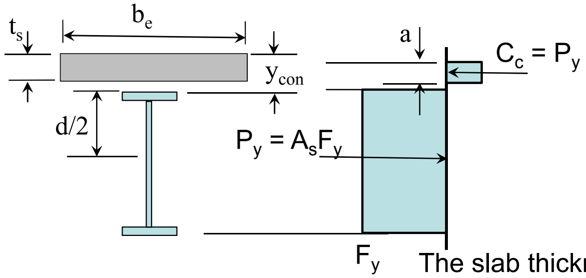
Preliminary Sizing – Step 1



Based upon the design moment, M_u, in kip-ft

$$A_s = \left[\frac{12M_u}{F_y\left(\frac{d}{2} + y_{con} - \frac{a}{2}\right)}\right]$$

The slab thickness, t_s, will typically be 8-10 in. depending on girder spacing. Pick “d” based upon L/d ratios and guess a=0.75t_s.

Preliminary Sizing – Step 2

- Once we have estimated the required steel area, we can proportion the section based upon local buckling restrictions and typical flange width to depth ratios (b_f/d).
- The web thickness is often selected based upon a target web slenderness (d/t_w) = 120~137 (remember 137 is the noncompact limit (λ_r) for Gr. 50 steel).
- Once the web is selected, we can determine the total area of the flanges $A_{f\text{-}total}$ = (A_s – dt_w).
- For a doubly-symmetric shape the two flange sizes will be equal. For a singly-symmetric shape the ratio of the bottom flange to top flange area is approximately 1.5.

Preliminary Sizing – Step 2 (continued)

- As guidelines in flange proportioning, AASHTO allows a minimum ratio of the compression flange width to girder depth of 1/6 (Section 6.10.2.2). This is actually a very slender girder. A better target value will be b_f/d = 0.25.
- Other criteria for flange sizing is the compact flange provisions. $\lambda = b_f/2t_f < 9.15 = \lambda_p$ (for grade 50 steel). Going slightly above 9.15 will typically not have a significant impact on the behavior; however Section 6.10.2.2 has an upper limit of 12 to control distortion during welding.
- If the flange is non-compact, flange local buckling needs to be evaluated.

Preliminary Sizing – Step 3

- The actual plate thickness values and practical widths will generally give a different steel area compared to Step 1.
- Assuming the neutral axis is still in the slab and selecting a value for f'_c, calculate “$a_{req'd}$” as follows:

$$a_{req'd} = \frac{A_sF_y}{0.85f'_cb_e}$$

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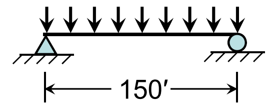
Preliminary Sizing – Step 4

- If "$a_{req'd}$" from Step 3 is less than t_s, the neutral axis is in the deck. If the value is significantly different than the value from step 1, recalculate the area of steel required in step 1 and iterate through the steps until "a" stabilizes. This will usually only take 1-2 iterations.
- If "$a_{req'd}$" is greater than t_s, the neutral axis is in the steel girder.
- Calculate the moment strength and adjust the girder proportions appropriately.

Preliminary Design Example

Consider the simply supported steel girder with L=150 ft. The girder is subjected to HL93 loading and other gravity loading that leads to a design moment of M_u =18000 k-ft. Develop trial sizes based upon the required design strength.



Concrete deck: f'_c = 4 ksi, t_s = 8 in., b_e = 120 in., 2" haunch

Steel: F_y = 50 ksi

Assume L/d~25: d=150′/25 = 6 ft = 72 in.

Selecting a 9/16" thick web plate results in a web slenderness d/t_w = 72″/0.56″ = 128 < 137 = λ_r (non-compact steel girder web)

Preliminary Design Example (Step 1): Approximate Steel Area

Assuming PNA in slab with a=0.75t_s=6 in.:

$$A_s = \left[\frac{12M_u}{F_y\left(\frac{d}{2} + y_{con} - \frac{a}{2}\right)}\right] = \left[\frac{12"/'x18000k'}{50ksi\left(\frac{72"}{2} + (8" + 2") - \frac{6"}{2}\right)}\right]$$

A_s =100 in^2 Total A_f = $A_s - Aweb$ =100 in^2 – 72" x 0.56" = 60 in^2

Assume a singly-symmetric section with A_{bf}=1.5A_{tf}

$$A_f = A_{bf} + A_{tf} = 1.5A_{tf} + A_{tf} = 60\ in^2:$$
$$A_{tf} = 24\ in^2, \qquad A_{bf} = 36\ in^2$$

Preliminary Design Example (step 2): Size Flanges

For top flange - targeting b_f/d = 0.25, results in a width of:
b_f = 0.25*72″ = 18″. Try 18″ x 1.125″
(flange slenderness = 18″/(2(1.125″))=8 < 9.15 = λ_p (compact)
Although this is smaller than the 24 in^2 estimated on the previous slide – with the singly symmetric section we will have a little larger moment arm. Plus this is a preliminary size.

For bottom flange – try b_f = 22″ and t_f = 1.625″ (A_{bf} = 35.8 in^2)

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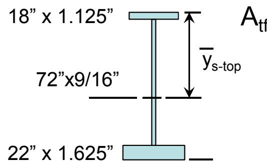
Preliminary Sizing (step 2) – section properties



A_{tf} = 20.25 in^2, A_{bf} = 35.8 in^2, A_{web} = 40.5in^2

$$\bar{y}_{s-top} = \frac{20.25x(1.13/2) + 40.5x(37.12) + 35.8x(73.94)}{96.6}$$

$\bar{y}_{s-top}$ =43.1″

A_{tot} = 96.6 in^2

Preliminary Sizing – Step 3

- Determine "$a_{req'd}$":

$$a_{req'd} = \frac{A_s F_y}{0.85 f_c' b_e} = \frac{96.6x50}{0.85x4x120} = 11.8" > 8"$$

PNA is in steel section

Compressive resultant in steel, C_s:

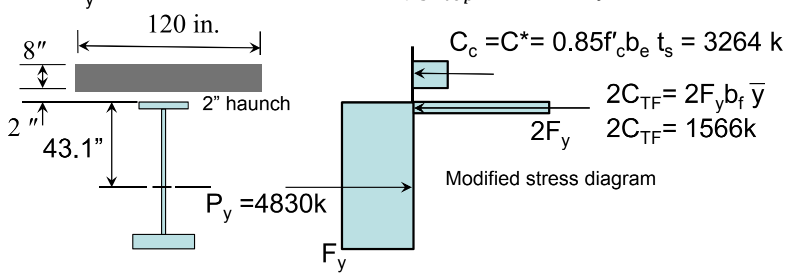
$$C_s = \frac{A_s F_y - C_c}{2} = \frac{96.6x50 - 0.85x4x120x8}{2} = 783k$$

Area of Steel in Compression:
A_{sc} = 783k/50ksi =15.7in^2 <20.24in^2 (A_{tf}),
PNA in top flange

Preliminary Sizing – Step 4

Distance from top of girder to PNA, $\bar{y}$ = 15.7in^2/18″=0.87″<t_f

P_y = 96.6in^{2}x50ksi = 4830k $\bar{y}_{s-top}$ =43.1″ $\bar{y}$ =0.87″



M_p = 4830 k (43.1″ – 0.87″/2) + 3264k (8"/2 + 2″+(0.87″/2))

M_p = 227,075 k-in = 18900 k-ft. > 18000 k-ft = M_u OK

Stability in Finished Bridge

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Stability in Finished Bridge

- We talked about the different stability limit states at the outset of this lecture; however our recent example did not consider any stability limit states.
- With the exception of the negative moment region, stability in the finished bridge is often not a concern.
- The reason for this is the location of both the PNA and the elastic neutral axis as well as the lateral/torsional restraint provided by the cured concrete deck. Local buckling in the positive moment region is not generally a problem.
- We will therefore not focus on the stability behavior in the finished bridge.

Stability During Construction

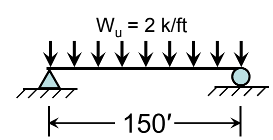
Stability During Construction

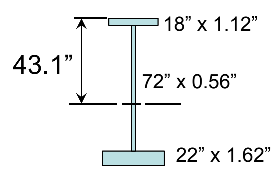
- While stability in the finished bridge of our example was not an issue, we do need to consider the behavior during construction.
- The critical stage for lateral torsional buckling will often be during placement of the concrete bridge deck.
- Stability is generally provided by the cross frames or diaphragms that are positioned along the length.
- The design moments will be the moments during deck placement.
- The AASHTO specification has a load factor of 1.4 that is applied to dead load during construction.

Construction Stage - Preliminary Sizing Example

A_{tf} = 20.25 in^2, A_{bf} = 35.8 in^2, A_{web} = 40.5in^2

Load Factor on Component Dead Load = 1.4





During Deck Placement:

$$M_u = \frac{W_u L^2}{8} = 5625\ k - ft.$$

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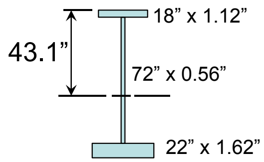
Construction Stage - Preliminary Sizing Example

A_{tf} = 20.25 in², A_{bf} = 35.8 in², A_{web} = 40.5in²



$$I_x = \frac{1}{12}(72)^3(0.56) + 40.5(43.1 - 37.12)^2$$
$$+20.3(43.1 - 1.12/2)^2$$
$$+ 35.8\left(72 + 1.12 + \frac{1.62}{2} - 43.1\right)^2$$
$$= 89500 in^4$$

$$S_{xc} = {}^{I_x}/_{c_{top}} = {}^{89500}/_{43.1} = 2077\ in^3$$

$$J = \sum \frac{1}{3}bt^3 = \frac{1}{3}[18x(1.12)^3 + 72(0.56)^3 + 22(1.62)^3] = 43.8\ in^4$$

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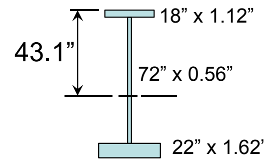
Construction Stage - Preliminary Sizing Example

A_{tf} = 20.25 in², A_{bf} = 35.8 in², A_{web} = 40.5in²



$$r_t = \frac{b_{fc}}{\sqrt{12\left[1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right]}}$$

$$r_t = \frac{18}{\sqrt{12\left[1 + \frac{1}{3}x\frac{42x0.56}{18x1.12}\right]}} = 4.41$$

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Construction Stage – Bracing Layout

- Although the girder has a non-compact web, we will start assuming elastic behavior and use the equation from the main body of AASHTO, which if you recall is for a slender web section.

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

- Design Construction Stress:

$$\frac{M_u}{S_{xc}} = \frac{5625x12}{2077} = 32\ ksi \le \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

Note: 32 ksi<35 ksi = 0.7F_y – therefore elastic as assumed

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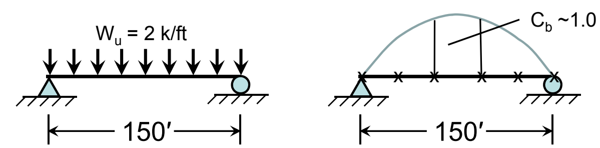
Construction Stage – Bracing Layout



$$32\ ksi = \frac{(1.0)(1.0)\pi^2(29000)}{\left(\frac{L_b}{4.41}\right)^2} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

L_b < 417 in. = 35 ft.

Number of unbraced segments = L/L_b = 150′/35′ = 4.3 (try 5)

Unbraced Length = 150′/5=30′ --- 4 intermediate X-frames

ie. cross frames at supports and 4 intermediate X-frames

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Construction Stage – Bracing Layout

- We can also check the elastic behavior using the expression for L_r from AASHTO Eq. 6.10.8.2.3-5:

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = \pi x 4.41 \sqrt{\frac{29000}{35}} = 399 in. < 417 in. = L_b$$

Elastic as assumed.

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Construction Stage – Bracing Layout

- We conservatively used the main body expressions that assume we have a slender web (we had λ=128<137=λ_r).
- Alternatively, we could use the equations in Appendix A6 that rely on both the St. Venant and Warping torsion.

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h} \left(\frac{L_b}{r_t}\right)^2}$$

- The expression is a little more involved to solve for L_b; however all we want to know is can we get by with one less cross frame to give L_b = 150′/4 = 37.5′=450″.

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Construction Stage – Bracing Layout

- With L_b=450 in., r_t=4.41 in., J=43.8 in^4, S_{xc}=2077 in^3, h=73.4 in. (distance between flange centroids), Appendix A6 Equation A6.3.3-8:

$$F_{cr} = \frac{1.0 x \pi^2 x 29000}{\left(\frac{450}{4.41}\right)^2} \sqrt{1 + 0.078 \frac{43.8}{2077 x 73.4} \left(\frac{450}{4.41}\right)^2} = 30.5 ksi < 32 ksi = \frac{M_u}{S_{xc}}$$

No Good – Stay with L_b=30 ft.

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System Buckling of Narrow Bridge Units (2-3 girder systems)

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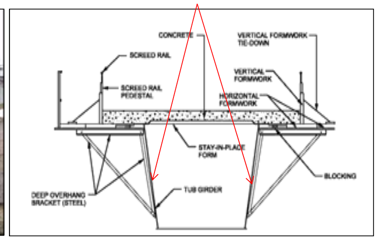
History of System Buckling Mode

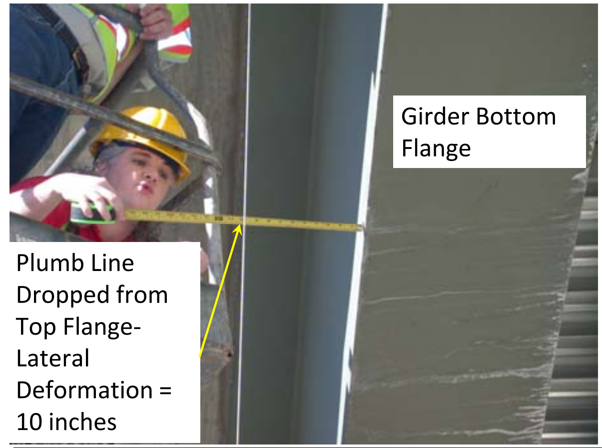
- Marcy Pedestrian Bridge – consisted of a single box girder with no top lateral truss. Girder had closely spaced internal K-frames (behaves very similar to a twin I-girder system)

Marcy tub girder behaved as twin I-girders



97

Global Buckling of Narrow Steel Units (con.)



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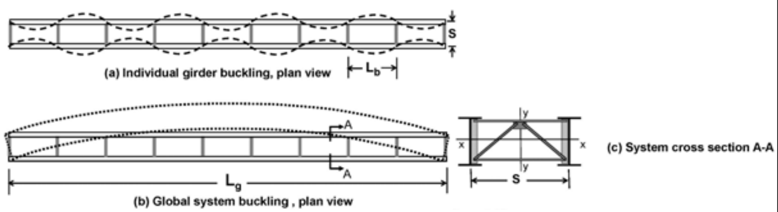
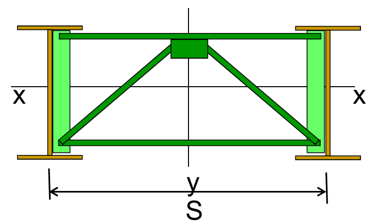
Global Buckling of Narrow Steel Units

- For a twin-girder system: L_b vs. L_g
  - Bracing spacing controls individual girder lateral-torsional buckling
  - Bracing size and spacing doesn't control system buckling



99

Global Buckling of Narrow Steel Units



AASHTO Currently limits moment to 50% of capacity predicted by this equation to avoid excessive 2nd order deformations

$$M_{crs} = \frac{2\pi}{L}\sqrt{EI_yGJ + \frac{\pi^2E^2I_y\left(I_yh^2 + I_xS^2\right)}{4L^2}} \quad \text{(exact)}$$

$$M_{crs} = \frac{\pi^2 SE}{L^2}\sqrt{I_yI_x} \quad \text{(simple)}$$

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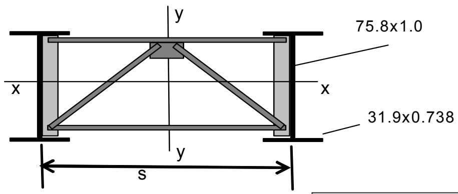
Global Buckling of Narrow Steel Units



Buckling Stress (ksi)			
	Girder Spacing S (in.)		
Analysis Type	80	109	150
ANSYS	21.31	28.56	38.68
Exact	21.40	28.73	39.21
Simple	20.70	28.22	38.83

Global Buckling of Narrow Steel Units

AASHTO eq. for system buckling during deck pour

$$M_{gs} = \frac{\pi^2 sE}{L^2}\sqrt{I_{eff} I_x}$$

AASHTO Eqn. 6.10.3.4.2-1

- Considering all of the girders across the width of the unit within the span, the sum of the largest total factored girder moments during deck placement should not exceed 50% M_{gs}.
- Should the sum of the moments exceed 50%, the designer can:
  - Add flange level lateral bracing
  - Revise the unit to increase system stiffness
  - Evaluate the amplified girder second-order displacements and verify they are within owner tolerances

Reference: Yura, J. A., T. Helwig, R. Herman, and C. Zhou. 2008. “Global Lateral Buckling of I-Shaped Girder Systems,” *Journal of Structural Engineering*, ASCE, Vol 134, No. 9, pp. 1487–1494.

Lecture L2 Recap

- Buckling limit states involve a loss of stiffness of a cross sectional element, an overall member, or an entire structural system.
- Local buckling modes include Local Flange Buckling, Local Web Buckling (Web Bend Buckling), and potentially effects around concentrated forces such as Web Crippling.
- Global Member Buckling for beams consists of Lateral Torsional Buckling.
- System Buckling modes where a few girders may buckle as a unit can occur in relatively narrow bridges.
- In many situations with global buckling, the structure provides significant warning of problems (lack of fit, etc.)

Lecture L2 Recap (continued)

- The vast majority of steel bridge girder systems are designed to be composite with the concrete bridge deck using welded shear studs to the top flange of the girders.
- In the positive moment region, the plastic neutral axis can be in one of three locations under gravity loading:
  - Concrete Deck
  - Top flange of steel girder
  - Web of steel girder
- In the negative moment region, composite action is not very efficient since the concrete deck is in tension.

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Lecture L2 Recap (Continued)

- In preliminary sizing, efficient proportions of the cross section can be obtained based upon a number of basic guidelines:
  - Typical span/depth ratios
  - Assumptions on PNA location for composite girder
  - Web slenderness between 120~137 (Gr. 50 steel)
  - Compression Flange/Depth ratio of approximately 0.25
- The spacing between cross frames and diaphragms is often controlled by lateral torsional buckling during casting of the construction deck.

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Thank You

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Polling Question 1

If the unbraced length of a girder becomes too large, the limit state that may control the capacity is:

a. Web Crippling
b. Web Bend Buckling
c. Flange Local Buckling
d. Lateral Torsional Buckling

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Polling Question 2

When evaluating the composite strength of I-shaped steel girders near midspan (gravity load), which of the following is NOT a possible location for the Plastic Neutral Axis?

a. Concrete bridge deck
b. Bottom flange of steel girder
c. Top flange of steel girder
d. Web of the steel girder

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Questions?

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Steel Bridge Design

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Event	Date	Handouts	Video	Quiz	Attendance
R1: Introduction To Bridge Engineering	N/A	Handouts	View Passcode: RZNS141	Pass Score: 80	N/A
R2: Introduction and History of AASHTO Bridge Design	N/A	Handouts	Available 9/11/2017 5:00 PM EDT	Available 9/11/2017 5:00 PM EDT	N/A
R3: Steel Material Properties	N/A	Handouts	Available 9/11/2017 5:00 PM EDT	Available 9/11/2017 5:00 PM EDT	N/A
R4: Loads and Analysis	N/A	Handouts	Available 9/11/2017 5:00 PM EDT	Available 9/11/2017 5:00 PM EDT	N/A
L1: Steel Bridge Fabrication	Oct 12 2017 1:30PM EDT	Handouts	Available 10/14/2017 5:00PM EDT	Available 10/14/2017 5:00PM EDT	Pending
L2: Plate Girder Design and Stability	Oct 19 2017 1:30PM EDT	Handouts	Available 10/21/2017 5:00PM EDT	Available 10/21/2017 5:00PM EDT	Pending
L3: Effects of Curvature and Skew	Oct 26 2017 1:30PM EDT	Handouts	Available 10/28/2017 5:00PM EDT	Available 10/28/2017 5:00PM EDT	Pending
L4: Fatigue and Fracture	Nov 2 2017 1:30PM EDT	Handouts	Available 11/04/2017 5:00PM EDT	Available 11/04/2017 5:00PM EDT	Pending
Intro To Steel Bridge Design - Final Exam	Nov 23 2017 8:00AM EST			Available 11/25/2017 5:00PM EST	

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- For Sessions R1 – R4, find access to recordings starting September 11. Recording access expires on November 23.
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- For Sessions R1 – R4, find access to quizzes starting September 11. Quizzes are due on November 23.
- For Sessions L1 – L4, find access to quizzes within two days after the live air date. Quizzes are due three weeks after the live session.
- A final exam will also be given.
- Quiz scores are displayed in the Course Resources table.

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  - Option 2: Watch the recording and pass the quiz.

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Thank You

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Survey at conclusion of webinar.

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