Loads and Flexural Design of Bridges (AASHTO LRFD 2017)

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S-8A SEng PRP Training Program

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AASHTO LRFD Limit States

- The AASHTO LRFD Specifications require consideration of several load combinations corresponding to the following limit states (Chapter 3):
  - STRENGTH LIMIT STATES (strength and stability)
  - SERVICE LIMIT STATES (stress, deformation, and cracking)
  - FATIGUE & FRACTURE LIMIT STATES (stress range)
  - EXTREME EVENT LIMIT STATES (earthquakes, wind, vehicle and vessel collision, among others)

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Load and Load Designations (3.4.1)

STRENGTH I :	normal vehicular use without wind
STRENGTH II :	owner special design/evaluation, permit vehicles without wind
STRENGTH III :	bridge exposed to design wind speed at the location (50 year, 7% chance)
STRENGTH IV :	very high dead-to-live load ratios (heavy bridges)
STRENGTH V :	normal vehicular use with 129 kph (80 mph) wind

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Load and Load Designations (3.4.1)

SERVICE I:	normal operational use of the bridge with 88 kph (55 mph) wind and nominal loads. Control cracking of reinforced concrete structures.
SERVICE II:	control yielding of steel structures and slip of connections due to live load.
SERVICE III:	control cracking in prestressed concrete superstructures
SERVICE IV:	control cracking in prestressed concrete substructures

SERVICE III & IV only apply to TENSION in PS members

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Load and Load Designations (3.3.2, page 3-8)

DD =	down drag
DC =	dead load of structural components and nonstructural attachments
DW =	dead load of wearing surfaces and utilities
EH =	horizontal earth pressure
EL =	accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
ES =	earth surcharge load
EV =	earth fill vertical pressure
BR =	vehicular braking force
CE =	vehicular centrifugal force
CR =	creep
CT =	vehicular collision force
CV =	vessel collision force

EQ =	earthquake
BL =	blast load
FR =	friction
IC =	ice load
IM =	vehicular dynamic load allowance
LL =	vehicular live load
LS =	live load surcharge
PL =	pedestrian live load
SE =	settlement
SH =	shrinkage
TG =	temperature gradient
TU =	uniform temperature
WA =	water load and stream pressure
WL=	wind load on live load
WS=	wind load on structure

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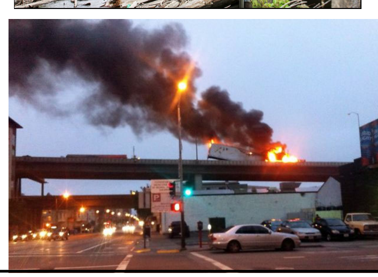
Other Loads?



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Other Loads?

Identify possible loads from AASHTO LRFD

Bridge Location

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Load Combinations and Load Factors (Table 3.4.1-1, page 3-14)

Has a range

Load Combination / Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME-I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	0.30	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

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Load Combination Factors (Table 3.4.1-1)

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	IC	CT	CV
STRENGTH-I	γ_p	1.	–	–	–
STRENGTH-II	γ_p	1.	–	–	–
STRENGTH-III	γ	-	–	–	–
STRENGTH-IV EH, EV, ES, DW	γ_p	.	–	–	–
DC ONLY	1.5				
STRENGTH-V	γ_p	1.	–	–	–
EXTREME EVENT-I	γ_p	γ	–	–	–
EXTREME EVENT-II	γ_p		1.00	1.00	1.00
SERVICE-I	1.00	1.	–	–	–
SERVICE-II	1.0	1.	–	–	–
SERVICE-III	1.00	0.	–	–	–
SERVICE-IV	1.00		–	–	–
FATIGUE-LL, IM & CE ONLY - I	–	[redacted]	–	–	–
FATIGUE-LL, IM & CE ONLY - II	–	0.	–	–	–

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS
STRENGTH-I	γ_p	1.75
SERVICE-I	1.00	1.00
SERVICE-III	1.00	0.80
FATIGUE-LL, IM & CE ONLY - I	–	[redacted]
FATIGUE-LL, IM & CE ONLY - II	–	0.75

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Load Factors for Permanent Loads, γ_p (Table 3.4.1-2)

Type of Load	Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

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

Range of load factors for dead loads must be considered

Only the two extremes needed

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Resistance Factors (5.5.4.2, p. 5-26)

	Standard Specs	LRFD 5.5.4.2
Flexure – RC	0.90	0.90
Flexure – PS	1.00	1.00
Shear – RC	0.85	0.90
Shear – PS	0.90	0.90
Compression	0.70 / 0.75	0.75
Bearing	0.70	0.70

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Load Factors, Service-I and Service-III Load Combinations

	Standardd Specs	LRFD Specs
DC	1.0	1.0
DW	1.0	1.0
LL (I)	1.0	1.0
LL (III)	1.0	0.8

where:

LL (I) = Live load → Service-I

LL (III) = Live load → Service-III for "tension in prestressed concrete structures with the objective of crack control"

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Bridge Load Types

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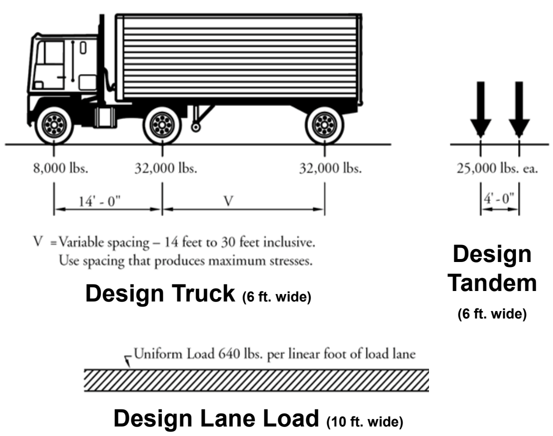
Design Vehicular Live Loads (3.6.1.2.1, p. 3-19)



Total Vehicular LL
(HL-93)

Design Truck
OR
Design Tandem
+
Design Lane Load

Tandem: heavy truck with short wheels

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Bridge Load Types

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Bridge Load Types

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Dynamic Load Allowance (3.6.2.1, p. 3-31)

The dynamic load allowance in Table 3.6.2.1-1 is an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles.

Table 3.6.2.1-1 Dynamic Load Allowance, *IM*.

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

Sources of dynamic effects on bridges
- Hammering at surface discontinuities
- Dynamic response of bridge as a whole

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Dynamic Load Allowance (3.6.2.1) (Impact)

For design of most bridge components for all limit states except fatigue
- The LRFD Specifications simply require a constant magnification (IM) of 33% to be applied to the design truck or design tandem only
- The magnification (IM) is not applied to the design lane load
- This simple approach is based on a study that found the most influential factor affecting dynamic impact is roadway surface roughness
- Commentary has more background

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Live Load Distribution Factors to Girders

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Common Superstructures (Table 4.6.2.2.1-1, p. 4-33)

SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	(b)
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	(c)
Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys	Cast-in-place concrete overlay	(f)
Precast Solid, Voided or Cellular Concrete Box with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	post tension (g)
Precast Concrete Channel Sections with Shear Keys	Cast-in-place concrete overlay	(h)
Precast Concrete Double Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	post tension (i)
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	post tension (j)
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	(k)

SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	post tension (j)
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	(k)

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Simplified Distribution Factors (4.6.2.2.1, p. 4-29)

To use the simplified distribution factors, the following conditions must be met:

- Width of deck is constant
- Number of beams, $N_b \geq 4$
- Beams are parallel and of the same stiffness
- The roadway part of the overhang, $d_e \leq 3.0$ ft.
- Curvature is less than limit in 4.6.1.2.4
- Section appears in Table 4.6.2.2.1-1

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Design Lanes

- Width of Design Lane = 12 ft. (LRFD 3.6.1.1, p. 3-19)
- No. of Design Lanes = w (clear roadway)/12; take the integer part of this ratio
- If traffic lane width is less than 12 ft., number of design lanes = number of traffic lanes.

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Unit Weights

- In absence of more precise information, Table 3.5.1-1 (LRFD p. 3-18) may be used.

Table 3.5.1-1—Unit Weights

Material		Unit Weight (kcf)
Aluminum Alloys		0.175
Bituminous Wearing Surfaces		0.140
Cast Iron		0.450
Cinder Filling		0.060
Compacted Sand, Silt, or Clay		0.120
Concrete	Lightweight	0.110 to 0.135
	Normal Weight with $f'_c \leq 5.0$ ksi	0.145
	Normal Weight with $5.0 < f'_c \leq 15.0$ ksi	$0.140 + 0.001 f'_c$
Loose Sand, Silt, or Gravel		0.100
Soft Clay		0.100
Rolled Gravel, Macadam, or Ballast		0.140
Steel		0.490
Stone Masonry		0.170
Wood	Hard	0.060
	Soft	0.050
Water	Fresh	0.0624
	Salt	0.0640
Item		Weight per Unit Length (klf)
Transit Rails, Ties, and Fastening per Track		0.200

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Distribution of Live Loads Per Lane, Moment in Interior Beams (Table 4.6.2.2.2b-1, p. 4-37)

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

Notes: 1) Units are in LANES and not WHEELS!
2) No multiple presence factor applied (tabulated equations)
4) May be different for positive and negative flexure locations
5) Use greater of 1 or 2 lanes loaded

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Longitudinal Stiffness Parameter (p. 4-32)

$$\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

Gives an indication of the relative stiffness between the beam (longitudinal) and deck (transverse)

For preliminary design, this term may be taken as 1.10

K_g = longitudinal stiffness parameter, in.^{4}, $= n(I + Ae_g^2)$ [LRFD Eq. 4.6.2.2.1-1]

where

n = modular ratio between beam and deck materials

$= \dfrac{E_c(\text{beam})}{E_c(\text{slab})}$ > 1, the inverse of ratio (n) for section properties since it is transforming beam to deck

A = cross-section area of the beam (non-composite section), in.^{2}

I = moment of inertia of the beam (non-composite section), in.^{4}

e_g = distance between the centers of gravity of the beam and deck, in.

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Distribution Factors for I-Beams – Shear (Table 4.6.2.2.3a-1, p. 4-43)

The live load distribution factor for shear for interior beams with 2 or more lanes loaded:

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

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Skew Bridges

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Distribution Factors for I-Beams: Moment with Skew (Table 4.6.2.2.2e-1, p. 4-41)

Bending moments in interior and exterior beams on skewed supports may be reduced using the following multiplier:

Use: $1 - c_1 (\tan\theta)^{1.5}$

where $c_1 = 0.25\left(\dfrac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.25}\left(\dfrac{S}{L}\right)^{0.5}$

Set $c_1 = 0$ when $\theta < 30°$

Set $\theta = 60°$ when $\theta > 60°$

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Distribution Factors for I-Beams – Shear with Skew (Table 4.6.2.2.3c-1)

Shear in exterior beams at the obtuse corner of the bridge may be reduced using the following multiplier:

$$\text{Use: } 1.0 + 0.20\left(\frac{12.0 L t_s^{\,3}}{K_g}\right)^{0.3} \tan\theta$$

This formula is valid for θ < 60°

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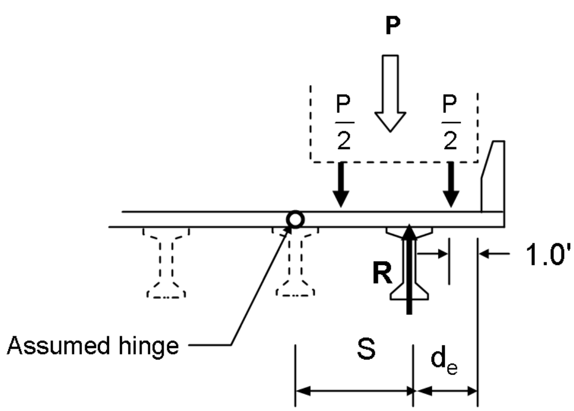
Lever Rule (4.6.2.2)



This approach is used for exterior girders only.

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Refined Methods of Analysis (4.6.3, p. 4-67)

4.6.3.1 General

Refined methods, listed in Article 4.4, may be used for the analysis of bridges. In such analyses, consideration shall be given to aspect ratios of elements, positioning and number of nodes, and other features of topology that may affect the accuracy of the analytical solution.

Nine methods are listed in Article 4.4 (p. 4-10), including

- Finite element method
- Finite difference method
- Grillage analogy method
- Yield line method

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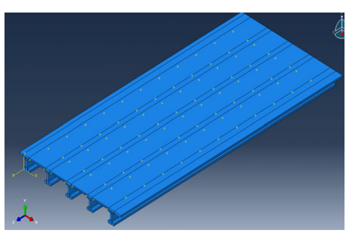
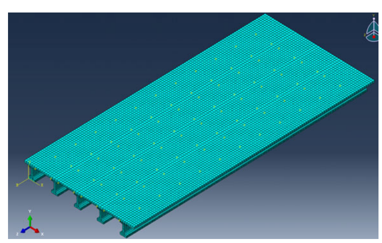
Finite Element Modeling and Evaluation, SH75 Bridge over Wilson Creek, McKinney, Texas





- Five girders with precast panels and CIP deck were modeled in ABAQUS CAE (2018) software.
- Using 3D deformable concrete elements.
- Prestressing and mild steel modeled as truss element.
- Thermal loading technique was used to apply prestress.

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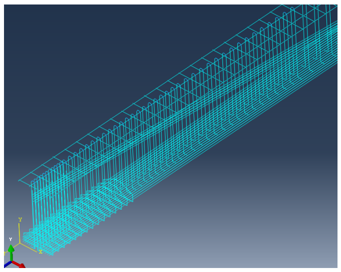
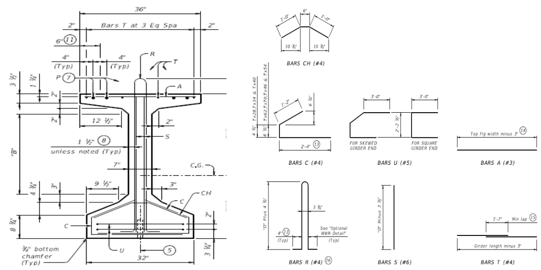
Finite Element Modeling (FEM)





- Each girder was modeled with 50 prestressing strands and 7 types of mild rebars.
- Material and section properties were defined accordingly.

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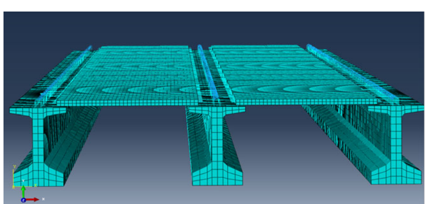
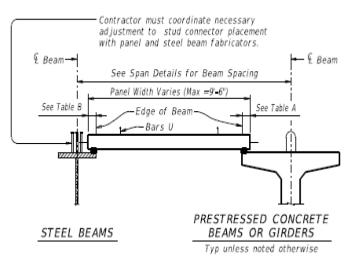
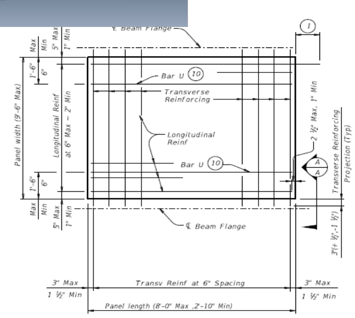
Finite Element Modeling (FEM)







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Design of Girders for Flexure

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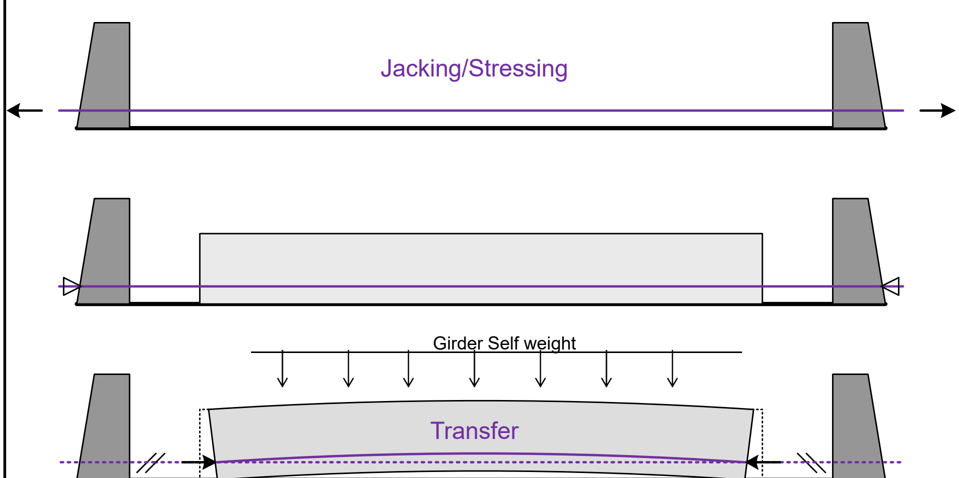
Construction Sequence



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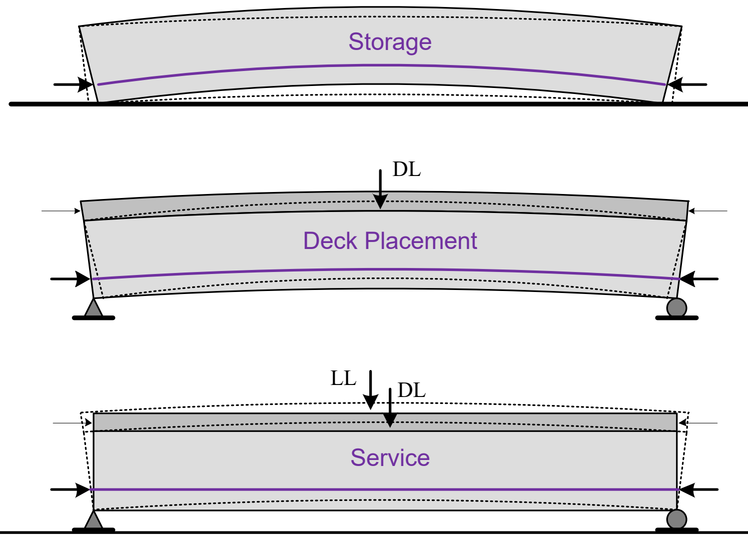
Loading Timeline



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Loading Timeline

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Loading Timeline

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Stress Limits: Prestressing Steel (Table 5.9.2.2.1, page 5-121)

YN1

Source: AASHTO (2017)

Table 5.9.2.2.1 **Stress Limits for Prestressing Tendons.** (Page 5-121)

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	0.70 f_{pu}	0.75 f_{pu}	—
At service limit state after all losses (f_{pe})	0.80 f_{py}	0.80 f_{py}	0.80 f_{py}
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	0.90 f_{py}	0.90 f_{py}	0.90 f_{py}
At anchorages and couplers immediately after anchor set	0.70 f_{pu}	0.70 f_{pu}	0.70 f_{pu}
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	0.70 f_{pu}	0.74 f_{pu}	0.70 f_{pu}
At service limit state after losses (f_{pe})	0.80 f_{py}	0.80 f_{py}	0.80 f_{py}

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YN1 Table 5.9.2.2.1, page 5-121

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Stress Limits: Conc. Comp. (Transfer)

YN2

5.9.2.3- Stress Limits for Concrete

5.9.2.3.1 —For Temporary Stresses before Losses—Fully Prestressed Components

5.9.2.3.1a – Compressive Stresses

The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be $\boxed{0.65\ f'_{ci}}$ (ksi).

5.9.2.3.1b – Tensile Stresses

The limits in Table **5.9.2.3.1b-1** shall apply for tensile stresses.

Source: AASHTO (2017)

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Stress Limits: Conc. Tension (Transfer)

YN3
YN4

Table 5.9.2.3.1b-1 Temporary Tensile Stress Limits in Prestressed Concrete Before Losses

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	• In precompressed tensile zone without bonded reinforcement	N/A
	• In areas other than the precompressed tensile zone and without bonded reinforcement	$\boxed{0.0948\lambda}$
		$= 3\sqrt{f_c'}\ (psi)$
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5\,f_y$, not to exceed 30 ksi.	$0.24\lambda\ \sqrt{f'_{ci}}$ (ksi)
	• For handling stresses in prestressed piles	$0.158\lambda\ \sqrt{f'_{ci}}$ (ksi)
Segmentally Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	

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YN2 0.65 f'ci
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YN3 insert
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YN4 lambda
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Stress Limits: Conc. Comp (Service)

Table 5.9.2.3.2a-1 Compressive Stress Limits in Prestressed Concrete at Service Limit State After Losses, Fully Prestressed Components.

Location	Stress Limit
• In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45 f'_c$ (ksi)
• In segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45 f'_c$ (ksi)
• Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling	$0.60\,\phi_w f'_c$ (ksi)

Source: AASHTO (2017)

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Stress Limits: Conc. Tension (Service)

YN5

Table 5.9.2.3.2b-1 Tensile Stress Limits in Prestressed Concrete at Service Limit State After Losses

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	$= 6\sqrt{f'_c}\ (psi)$
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.19\lambda\sqrt{f'_c} \leq 0.6$ (ksi)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.0948\lambda\sqrt{f'_c}$ (ksi)
	• For components with unbonded prestressing tendons	No tension
Segmentally Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	

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YN5 insert lambda
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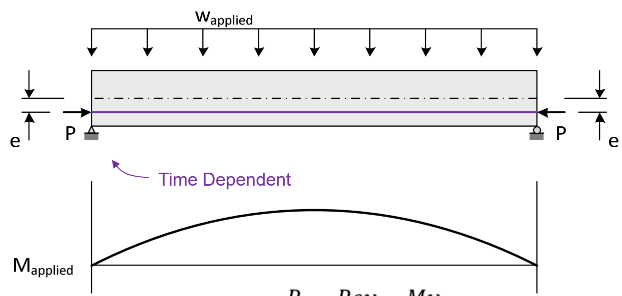
Stress Analysis



$$f = \frac{P}{A_g} \pm \frac{Pey}{I_g} \pm \frac{My}{I_g}$$

Gross vs. Transformed Section Properties

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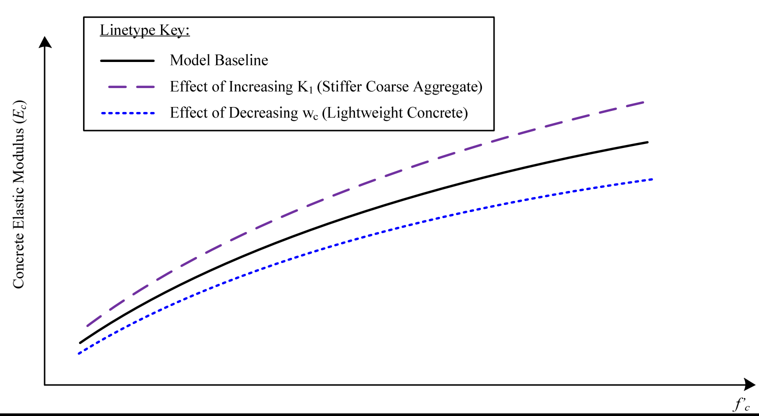
Concrete Properties: Elastic Modulus

AASHTO 5.4.2.4, p. 5-19

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'}$$



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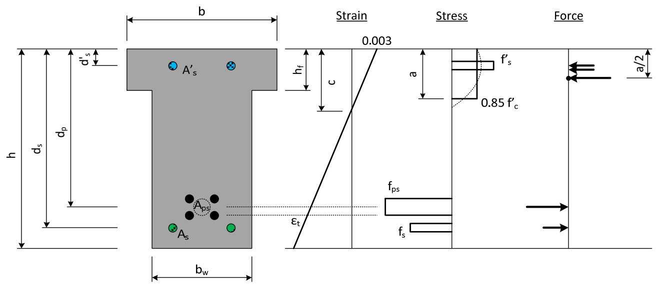
YN6

Nominal Moment Capacity



AASHTO 5.6.3.2, p. 5-39)

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right)$$
$$+ \alpha_1 f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

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YN7

Prestressing Steel Stress at Ultimate

AASHTO 5.6.3.1.1, p. 5-36

For bonded tendons:

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

For low relaxation strands, k = 0.28

To locate the neutral axis:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - \alpha 1\, f'_c(b - b_w)h_f}{\alpha 1 f'_c\beta_1 b_w + kA_{ps}\left(\frac{f_{pu}}{d_p}\right)}$$

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YN6 change 0.85 to alpha1
Yazdani, Nur, 9/26/2018

Slide 48

YN7 change 0.85
Yazdani, Nur, 9/26/2018

Resistance Factors

YN8

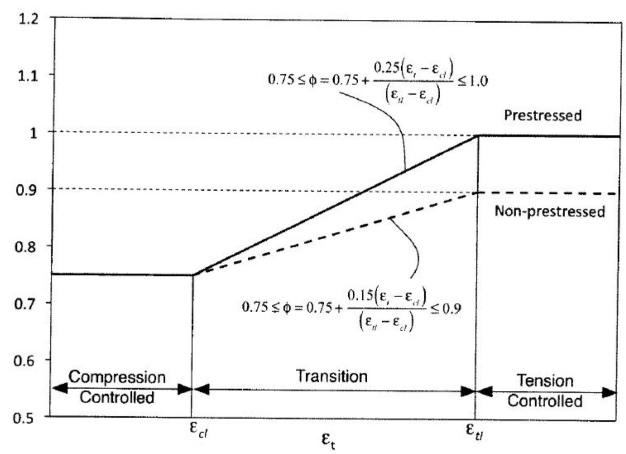

Figure C5.5.4.2.-1—Variation of ϕ with Net Tensile Strain ε_t for Nonprestressed Reinforcement and for Prestressing Steel

AASHTO 5.5.4.2, p. 5-30

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Reinforcement Limits

- Maximum Reinforcement (5.6.3.3, p. 5-41)
  - No limit; has to be tension controlled
  - Effectively controlled by resistance factor, ensuring ductility
- Minimum Reinforcement

$$\phi M_n \ge Min \begin{cases} M_{cr} \\ 1.33 M_u \end{cases}$$

- Control of Cracking by Distribution of Reinforcement (5.6.7 p. 5-54)

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YN8 change fig. See page 5-30 AASHTO

Yazdani, Nur, 9/26/2018

Deflection

- Stiffer girders yield more durable decks
- Criteria for Deflection (AASHTO 2.5.2.6.2, p. 2-11) [optional]
  - All design lanes loaded
  - Consider stiffness of entire superstructure, including barriers, etc.
  - Use elastic methods; calculate I_e if section is cracked (AASHTO 5.6.3.5.2, p. 5-43)
  - Recommended limit: L/800

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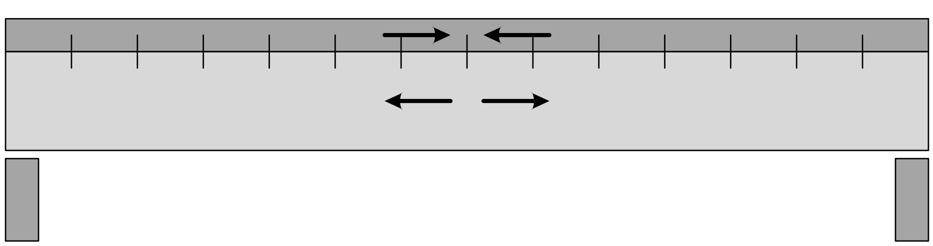
Composite Sections

AASHTO 5.7.4, p. 5-77

Design for interface shear



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Flexural Design Sequence

1. Estimate loads and span arrangement
2. Estimate cross section and material properties
3. Arrange prestressing to satisfy service stresses
   - If too much prestressing needed, use larger section, or increase number of girder lines
4. Consider draping/debonding to meet stress limits at transfer and end of span
5. Confirm ultimate capacity
6. Check reinforcing limits

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Points to Consider

- Service vs. Strength Limit States
- Sequence of construction/loading
- Stress analysis, including consideration for losses
- Prestress loss methods based on elastic analysis
- Girder design usually controlled by service stresses

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Example 1

A highway bridge uses steel girders.

Design Code:
AASHTO LRFD Bridge Design Specifications, 2017.

Design Data:
- Total service dead load moment (DC): 1,200 kip-ft
- Total service load moment including dynamic load allowance (LL + IM): 850 kip-ft
- Total service dead load moment due to wearing surface (DW): 100 kip-ft

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Example 1

The design moment (kip-ft) using the Strength 1 loading combination with maximum load factors is most nearly:

a) 2,912
b) 3,138
c) 3,438
d) 3,684

Solution:

$M_u = 1.25(1200) + 1.5(100) + 1.75(850)$ (Table 3.4.1-1, 3-15, 3.4.1-2, 3-16)

$= 3138\ kip - ft = 4255\ kN - m$

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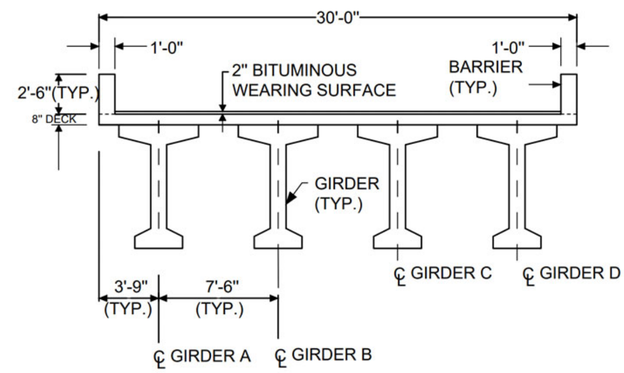
Example 2

Figure shows a bridge superstructure cross section. The bridge is a single-span structure with precast, prestressed concrete girders, a cast-in-place concrete deck, and a bituminous wearing surface. Barriers are cast in place.



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Example 7 Continued

Design Code: *AASHTO LRFD Bridge Design Specifications, 2017*

Design Data

- Concrete unit weight 150 pcf (precast and cast-in-place concrete)
- Wearing surface unit weight 140 pcf kcf
- Bridge skew 0°
- Bridge length 120 ft (centerline of bearing to bearing)

Girder properties

$f'c$ = 7 ksi

Weight = 0.60 kip/ft

I = 265,320 in^4

A = 570 in^2

e_g = 28 in

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Example 7 Continued

Cast-in-place properties:

f'_c = 4 ksi

Assume 36 – 0.5 in. diameter low relaxation 270 ksi strands.

Eccentricity =

Assumptions:

- Bridge barrier and wearing surface are applied evenly to all girders
- Ignore design tandem loading
- Superstructure is a conventionally redundant system
- Bridge is considered operationally essential
- Bridge is conventional design

Example 2

Solution:

$W_{girder} = 0.60\ kip/ft\ (given)$

$W_{deck} = \left[\left(\frac{7'-6"}{2} * 2\right)\right]\left(\frac{8}{12}\right)(0.150) = 0.75\ kip/ft$

$W_{barrier} = \frac{2\ barriers}{4\ girders}[(2'\ 6")(1)](0.15) = 0.19 kip/ft$

$W_{DC} = 0.6 + 0.75 + 0.19 = 1.54\ kip/ft$

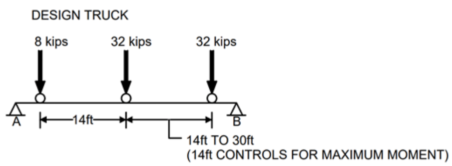
$M_{DC} = \frac{wL^2}{8} = \frac{1.54(120)^2}{8} = 2772\ kip - ft$

Determine the wearing surface moment for Girder A:

$W_{DW} = \frac{[30'-2(1')]}{4\ girders}\left(\frac{2}{12}\right)(0.140) = 0.163\ kip/ft$ (Table 3.5-1-1, page 3-19)

$M_{DW} = \frac{wL^2}{8} = \frac{(0.163)(120)^2}{8} = 293\ kip - ft$

Example 2



Note: It is sufficiently accurate to place the center axle at midspan.

$\sum M_B = 0 = -(32)(60' - 14') - (32)(60') - (8)(60' + 14') + R_A(120')$

$R_A = \frac{3,984}{120} = 33.2\ kip$

$\sum M_{CL} = 0 = 33.2(60') - 8(14') - M_{truck} = 0$

$M_{truck/lane} = 1880\ kip - ft/lane$

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Example 2

- Design lane

  $W_{lane} = 0.64\ kip/ft\ (over\ 3\ m\ width)$ (Art. 3.6.1.2.4, 3-23)

  $M_{lane/lane} = \frac{wL^2}{8} = \frac{0.64(120\ \ )^2}{8} = 1152\ kip - ft$

- Dynamic Load Allowance (IM)

  Strength 1 limit state for girder => IM =33% (Art 3.6.2, 3-31)

- $IM_{factor\ for\ truck} = \left(1 + \frac{33}{100}\right) = 1.33$

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Example 2

- Application of design vehicular live loads (Art 3.6.1.3, 3-26)

Outside wheel of design truck placed 0.6 m from edge of design lane for girders.

$M_{LL+IM/lane} = 1880(1.33) + 1152 = 3652\ kip - ft/lane \ \approx 4951\ kN - m/lane$

Example 2

$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{2.75}{9.1} = 1.072$

$f_c'(girder) = 7.0\ ksi$ $\qquad W_{girder} = 0.150\ kcf$

$f_c'(C.I.P.) = 4.0\ ksi$ $\qquad W_{C.I.P.} = 0.150\ kcf$

$E_{girder} = 33000 K_1 W_c^{1.5}\sqrt{f_c'} = 33{,}000(1.0)(0.150)^{1.5}\sqrt{7.0} = 5072\ ksi$ (Eq. C5.4.2.4-2, 5-20)

$E_{C.I.P.} = 33000(1.0)(0.150)^{1.5}\sqrt{4.0} = 3834\ ksi$

$n = \frac{E_B}{E_D} = \frac{5{,}072}{3{,}834} = 1.32$ (Eq.4.6.2.2.1-2, 4-32)

Example 2

$K_g = n(I + Ae_g^2)$

$= 1.32(265{,}320 + 570(28)^2 = 940{,}104\ in^4$

One design lane loaded: $g_{int} = 0.06 + \left(\frac{S}{14}\right)^{0.4} + \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$,

(Table 4.6.2.2.2b-1, 4-37)

Two or more design lanes loaded: $g_{int} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$

Applicability check

$3.5 \leq S = 7.5' < 16.0$ OK $N_b = 4$ OK

$4.5 \leq t_s = 8" \leq 12.0$ OK $10{,}000 \leq K_g = 940{,}104\ in^4 \leq 7{,}000{,}000$

$20 \leq L = 120' < 240$ OK

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Example 2

$g_{int} = 0.06 + \left(\frac{7.5}{14}\right)^{0.4} \left(\frac{7.5}{120}\right)^{0.3} \left(\frac{(940{,}104)}{12.0\ (120)(8^3)}\right)^{0.1} =$
$0.408\ one\ lane$

$g_{int} = 0.075 + \left(\frac{7.5}{19.5}\right)^{0.4} \left(\frac{7.5}{120}\right)^{0.3} \left(\frac{(940{,}104)}{12.0\ (120)(8^3)}\right)^{0.1} =$
$0.586\ two\ lane$

$g_{int} = 0.586$

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Example 2

For strength limit states:

$\gamma_{DC} = 1.25$ max, $\gamma_{DW} = 1.5$ max (Tables 3.4.1-1, 3-15, and 3.4.1-2, 3-16)

For Strength I Limit State:

$\sum \gamma_i Q_i = [\gamma_P(DC + DW) + 1.75(LL + IM)]$

$M_u = (1.25(2772) + 1.5(293) + 1.75(0.842)(3652)]$

$= 9285\ kip - ft \approx 12588\ kN - m, \qquad at\ midspan\ of\ Girder\ A$

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Example 2

$\mathrm{A_{ps}} = 36 * 0.153 = 5.508$ sq. in. (for 0.5 in, low lax),

$\mathrm{d_p} = 75.7$ in.

$$c = \frac{5.508\ (270)}{0.85(4)(0.85)(108) + 0.28(5.508)\dfrac{270}{75.7}} = 5.54 \text{ in.}$$

a = depth of stress block = 0.85(4.68) = 3.98 in.< 8 in.

So, beam behaves as rectangular!

Av. stress in strands = $270\left(1 - 0.28\frac{4.68}{75.78}\right) = 265$ ksi

Moment capacity, M_u = $5.508(265)\left(75.78 - \frac{3.98}{2}\right) = 8986\ k - ft.$

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Example 2

Check tension-controlled: $\epsilon_t = \frac{78-4.68}{4.68} * 0.003 = 0.047 > 0.005, OK!$

$$\emptyset = 1.0$$

Design moment capacity, M_r = 8986 k-ft = 12183 kN-m > M_u = 12588 kN-m,

NG!

Change design to 40 strands.

Still needed: prestress losses; service limit state checks; minimum steel; end and transfer points; shear design, interface shear, anchorage zone

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Questions?

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