Bridge Deck Design (AASHTO LRFD 2017)

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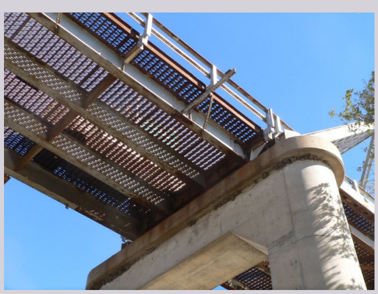
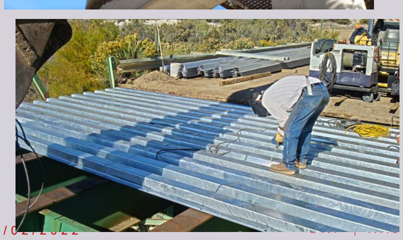
Design of Bridge Decks



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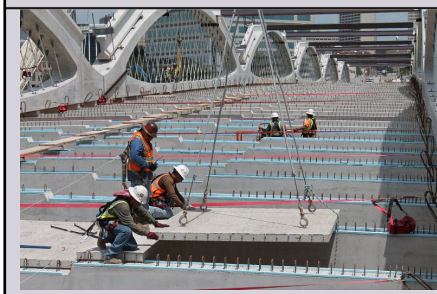
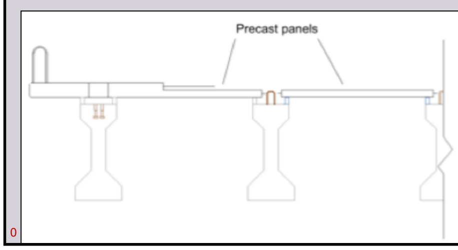
Design of Bridge Decks





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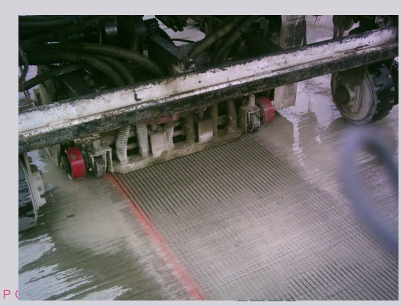
Design of Bridge Decks



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Design of Bridge Decks

- ✓ Deck replacement is impractical and almost impossible to do without closing the entire bridge. Therefore, it is always a good strategy to be conservative and allow for reserved capacity during design.
- ✓ Subjected to static and dynamic loads, thermal gradients, and creep and shrinkage effects. Proper consideration should be given to these effects to prevent cracking and deterioration.

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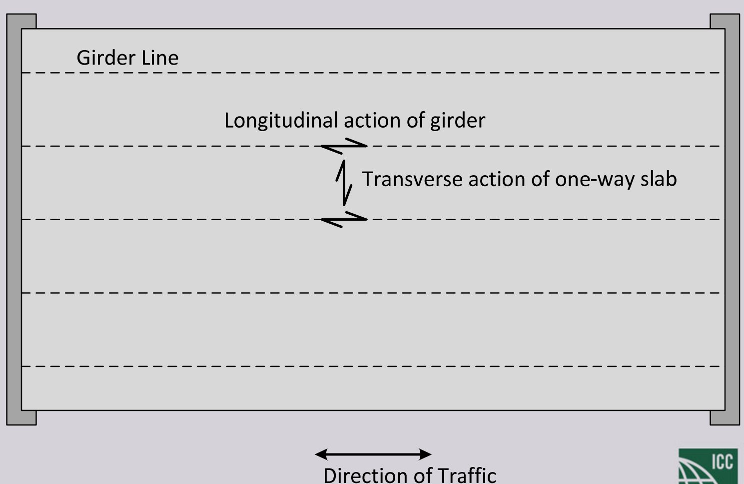
The Structural System



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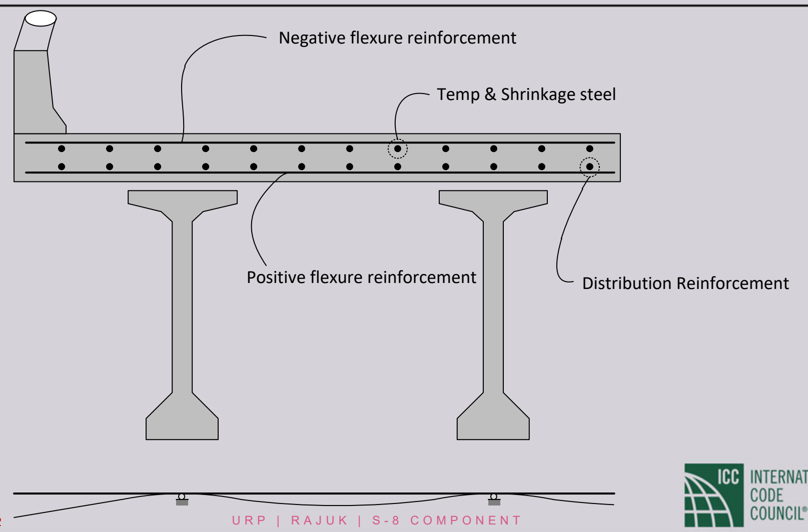
The Structural System



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AASHTO LRFD Bridge Design Specifications; Chapter 9: Decks

- Minimum deck thickness = 7 in. (p. 9-7)
- All previous limit states apply.
- Methods of deck analysis (p. 9-6).
- Minimum covers in AASHTO Table 5.10.1-1 (p. 5-165).
- For decks with less than 25 deg. skew, place primary rebars in the skew direction. Otherwise, place it normal to the supporting beams.

Table 5.10.1-1—Cover for Unprotected Main Reinforcing Steel (in.)

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above	
• Up to No. 11 bar	1.5
• No. 14 and No. 18 bars	2.0
Bottom of cast-in-place slabs	
• Up to No. 11 bar	1.0
• No. 14 and No. 18 bars	2.0
Precast soffit form panels	0.8
Precast reinforced piles	
• Noncorrosive environments	2.0
• Corrosive environments	3.0
Precast prestressed piles	2.0
Cast-in-place piles	
• Noncorrosive environments	2.0
• Corrosive environments	
○ General	3.0
○ Protected	3.0
• Shells	2.0
• Auger-cast, tremie concrete, or slurry construction	3.0
Precast concrete box culverts	
• Top slabs used as a driving surface	2.5
• Top slabs with less than 2.0 ft of fill not used as a driving surface	2.0
• All other members	1.0

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Concrete Covers
(different than BNBC-2020 and ACI 318!)

- AASHTO 5.10.1 (p. 5-165).
- Minimum cover is 1.0 in. always.
- For unprotected steel, use minimum covers in Table 5.10.1-1, modified for w/c ratio (p. 5-164).
  - For w/c ratio ≤ 0.4, modify by 0.8
  - For w/c ratio ≥ 0.5, modify by 1.2
- For post-tensioned ducts, use
  - Same as main reinforcing
  - ½ of duct diameter
  - Table 5.10.1-1
- For epoxy coated rebars, use interior exposure from Table 5.10.1-1 (p. 5-185).

> **5.10.1—Concrete Cover**
>
> Cover for unprotected prestressing and reinforcing steel shall not be less than that specified in Table 5.10.1-1 and modified for *W/CM* ratio, unless otherwise specified either herein or in Article 5.12.5.
>
> Modification factors for *W/CM* ratio shall be the following:
>
> - For *W/CM* ≤ 0.40 .... 0.8
> - For *W/CM* ≥ 0.50 .... 1.2

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Deck Analysis Methods
(AASHTO LRFD)

- Approximate Methods
  - Strip Method (4.6.2.1.1, p. 4-22)
    - Design aids allowed for decks with precast elements
  - Empirical Method (9.7.2, p. 9-8)
- Refined Methods (4.6.3.2, p. 4-67)
- Overhang Design (9.7.1.5)

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Strip Method - Overview

- Model slab strip as continuous beam (p. 4-26)
- Analyze to find moment at critical sections
- Determine width of strip sharing load effect
- Design as one-way concrete slab

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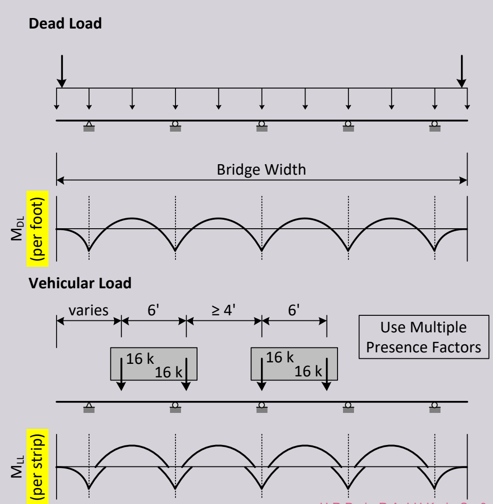
Strip Method – Analysis



LL moments can be found from AASHTO App. A4 (p. 4-98), Deck Slab Design Table

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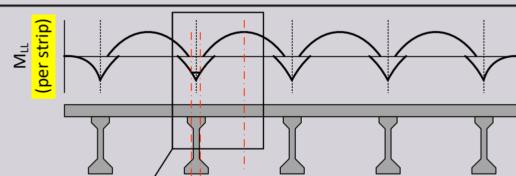
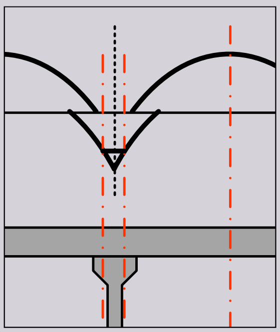
Strip Method – Critical Section





- AASHTO 4.6.2.1.6 (p. 4-22).
- For precast I-shaped girders, critical section for negative moment and shear is: $b_f/3 \leq 15$ in. from girder centerline (p. 4-27). For monolithic construction, precast boxes and stemmed beams, use section at the support face.
- Use beam theory for continuous beams and assume infinite rigidity for supporting elements.
- Apply maximum ±moments to all ± moment sections (p. 4-22).

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Location of Negative Moment

4.6.2.1.6—Calculation of Force Effects

The strips shall be treated as continuous beams or simply supported beams, as appropriate. Span length shall be taken as the center-to-center distance between the supporting components. For the purpose of determining force effects in the strip, the supporting components shall be assumed to be infinitely rigid.

The wheel loads may be modeled as concentrated loads or as patch loads whose length along the span shall be the length of the tire contact area, as specified in Article 3.6.1.2.5, plus the depth of the deck. The strips should be analyzed by classical beam theory.

The design section for negative moments and shear forces, where investigated, may be taken as follows:

- For monolithic construction, closed steel boxes, closed concrete boxes, open concrete boxes without top flanges, and stemmed precast beams, i.e., Cross-sections (b), (c), (d), (e), (f), (g), (h), (i), and (j) from Table 4.6.2.2.1-1, at the face of the supporting component,
- For steel I-beams and steel tub girders, i.e., Cross-sections (a) and (c) from Table 4.6.2.2.1-1, one-quarter the flange width from the centerline of support,

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Strip Method – Strip Width (p. 4-24)

Table 4.6.2.1.3-1—Equivalent Strips

Type of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (in.)
Concrete:		
• Cast-in-place	Overhang	$45.0 + 10.0X$
	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
• Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
• Precast, post-tensioned	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$

Source: AASHTO (2017)

X = distance of load from point of support
S = spacing of supports

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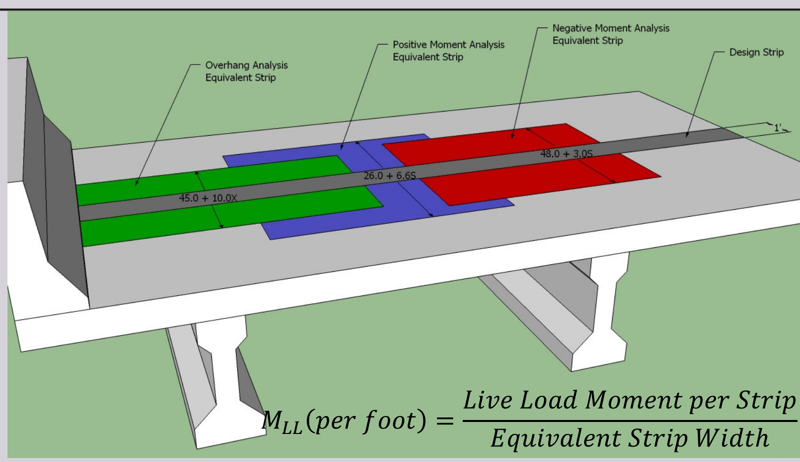
Strip Method – Strip Width



$$M_{LL}(per\ foot) = \frac{Live\ Load\ Moment\ per\ Strip}{Equivalent\ Strip\ Width}$$

LL Shortcut: AASHTO Table A4.1(p. 4-98)

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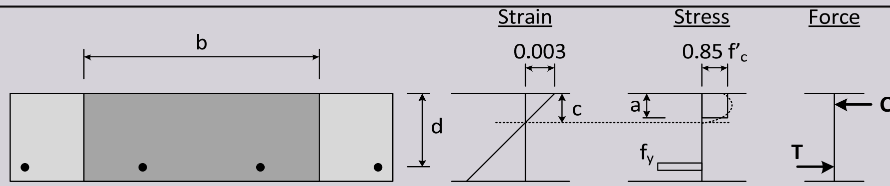
Strip Method – Strength Design

- Design Top and Bottom Transverse Reinforcement based on Factored Positive and Negative Moments.
  - » Include IM Factor.
  - » Include Multiple Presence Factors.

Strip Method – Strength Design



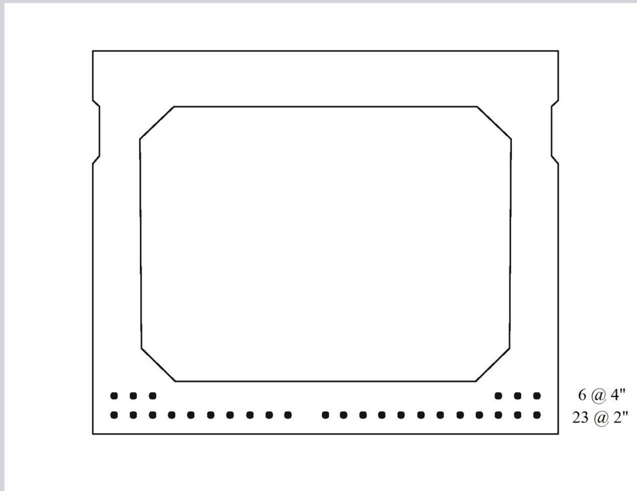
$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$a = \beta_1 c$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$A_s = \frac{A_{bar}}{s}$$

AASHTO Cover Provisions Satisfied?



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Rebar Spacing

5.10.3.2—Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not be greater than the lesser of the following:

- 1.5 times the thickness of the member; or
- 18.0 in.

The maximum spacing of spirals, ties, and temperature shrinkage reinforcement shall be as specified in Articles 5.10.4, 5.10.5, and 5.10.6.

5.10.3—Spacing of Reinforcement

5.10.3.1 Minimum Spacing of Reinforcing Bars

5.10.3.1.1—Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- 1.5 times the nominal diameter of the bars;
- 1.5 times the maximum size of the coarse aggregate; or
- 1.5 in.

5.10.3.1.2—Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- the nominal diameter of the bars;
- 1.33 times the maximum size of the coarse aggregate; or
- 1.0 in.

5.10.3.1.3—Multilayers

Except in decks where parallel reinforcement is placed in two or more layers, with clear distance between layers not exceeding 6.0 in., the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than 1.0 in. or the nominal diameter of the bars.

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Minimum (Primary) Reinforcement

AASHTO 5.6.3.3

$$\phi M_n \geq \text{least of} \begin{cases} M_{cr} \\ 1.33 M_u \end{cases}$$

$$M_{cr} = \frac{f_r I_g}{y_t}$$

I_g – *gross moment of inertia*

y_t – *neutral axis distance to tension face*

f_r – *modulus of rupture* $= 0.37\sqrt{f_c'}$ (ksi)

Upper bound

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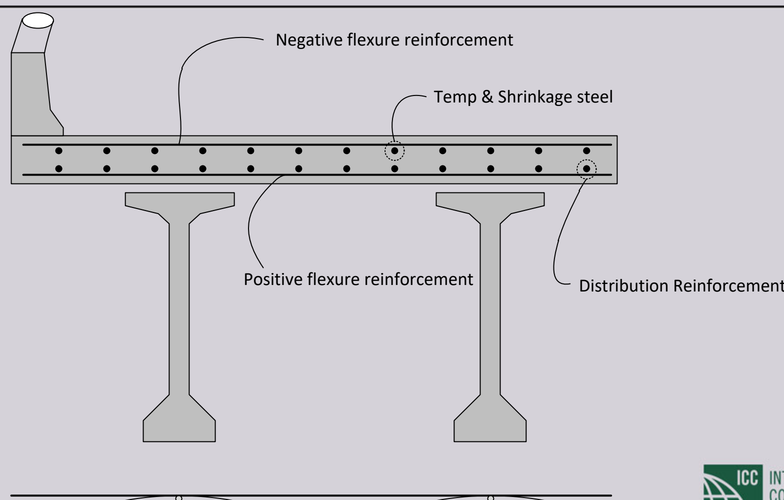
Distribution Reinforcement



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Distribution Reinforcement

AASHTO 9.7.3.2, p. 9-12

- Bottom of slab in secondary direction
  - » Engages additional primary positive moment reinforcement
- Based on a percentage of primary positive moment reinforcement
- When primary reinforcement is perpendicular to traffic:

$$\frac{A_{s-distribution}}{A_{s-primary\ positive}} = max \begin{cases} \frac{220}{\sqrt{S}}\% \\ 67\% \end{cases}$$

S − effective slab span (~ girder spacing) see AASHTO 9.7.2.3

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Temp. & Shrinkage Reinforcement

- Minimum reinforcement in both directions
- Usually only controls on top in secondary direction, distribution steel in bottom controls.

$$A_s\left(\frac{in^2}{ft}\right) \geq \frac{1.30\overbrace{bh}^{\text{Cross Sectional Area}}}{\underbrace{2(b+h)}_{\text{Drying perimeter}}f_y} \quad \text{where } 0.11 \leq A_s \leq 0.60$$

AASHTO 5.10.6, p. 5-170

- Bar spacing shall not exceed:

$$\text{least} \begin{cases} 3 * \text{slab thickness} \\ 18" \end{cases}$$

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Temp. & Shrinkage Reinforcement

Epoxy coated rebars

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Empirical Method – AASHTO 9.7.2 (p. 9-8)

- Based on research and FEM verification
- Load resistance mechanism – internal arching action
- No analysis required
- Isotropic reinforcement – same both directions in each layer
- Does NOT apply to overhang design

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Empirical Method

- Many design conditions must be met: diaphragms, fully cast in place and water cured, uniform depth, and others (p. 9-10)………
- Steel spacing ≤ 18 in., at least Grade 60 steel.
- Bottom layer, each way: 0.27 in^2/ft.
  - » ~ #5 bars at 13.5 in.
- Top layer, each way: 0.18 in^2/ft.
  - » ~ #4 bars at 13 inches
- May not give as much "feel" for the structure

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Overhang Design – AASHTO A13.4

- 3 design cases (AASHTO A13.4.1, p. 13-25)
  - » Horizontal Vehicle Collision (Extreme Event II)
  - » Vertical Collision (Extreme Event II. p. 3-10)
  - » DL and LL on Overhang (Strength I)
- 3 design sections
  - » Inside of parapet (A)
  - » Critical section outside exterior girder (B)
  - » Critical section inside exterior girder (C)

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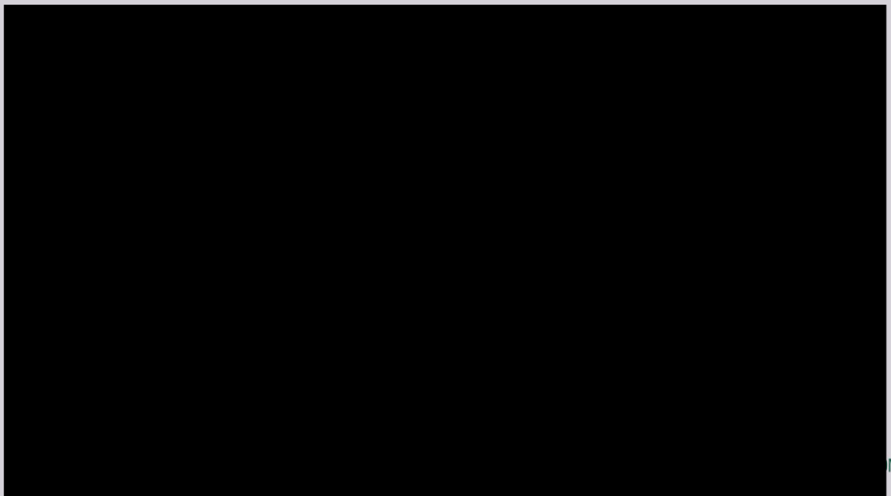
Overhang Design – AASHTO A13.4



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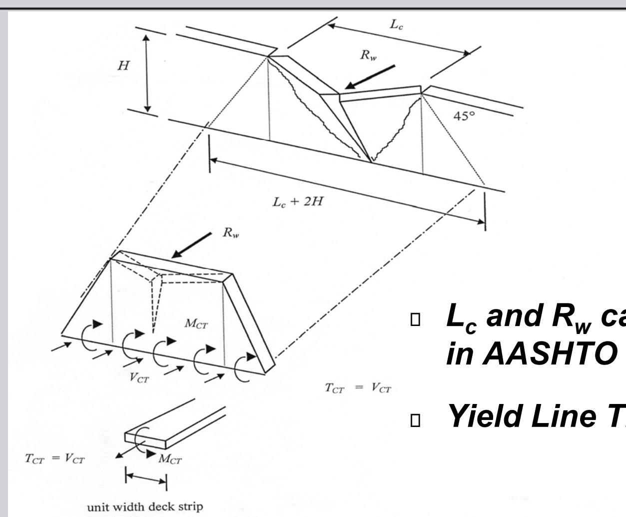
Barrier Capacity – Yield Line 1



- L_c and R_w calculated in AASHTO A13.3.1
- Yield Line Theory

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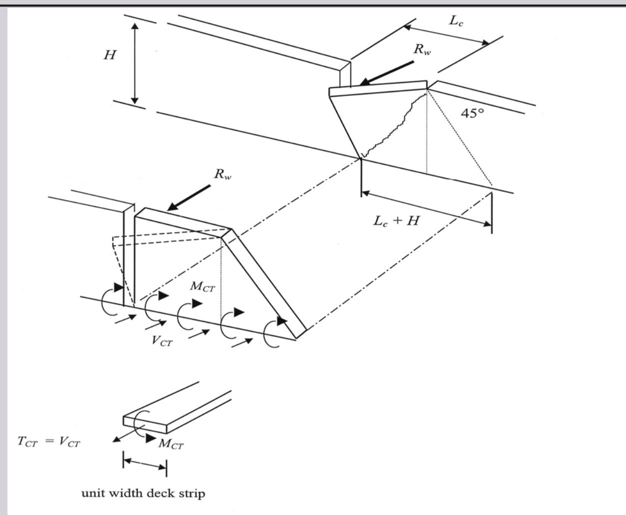
Barrier Capacity – Yield Line 2



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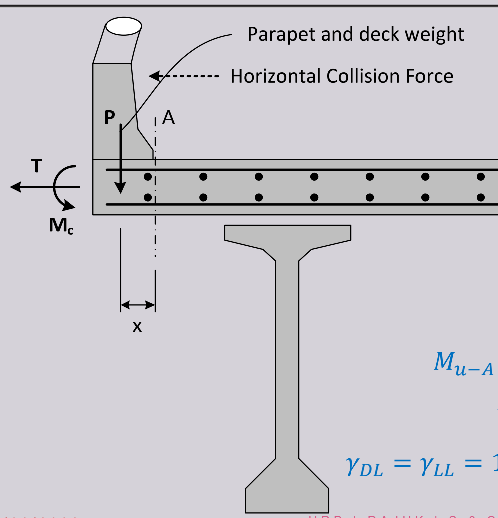
Case 1 – Section A



M_c: *Moment capacity of barrier about horizontal axis parallel to traffic*

$$T = \begin{cases} \textit{Yield Line 1:} \dfrac{R_w}{L_c + 2H} \\ \textit{Yield Line 2:} \dfrac{R_w}{L_c + H} \end{cases}$$

$$M_{u-A} = \gamma_{LL} M_c + \gamma_{DL} P x$$

$$T_{u-A} = \gamma_{LL} T$$

$$\gamma_{DL} = \gamma_{LL} = 1.0 \quad (\textit{Extreme Event})$$

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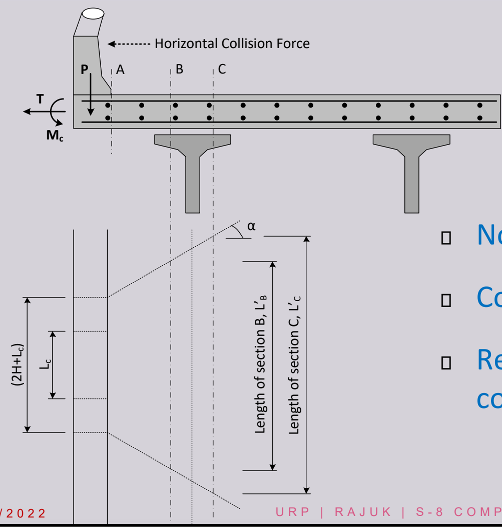
Case 1 – Section B & C



$$M'_B = M_c \frac{(L_c + 2H)}{L'_B}$$

$$T'_B = T \frac{(L_c + 2H)}{L'_B}$$

- No Spec guidance for α
- Could probably use 45 degrees
- Recommend 30 degrees conservatively

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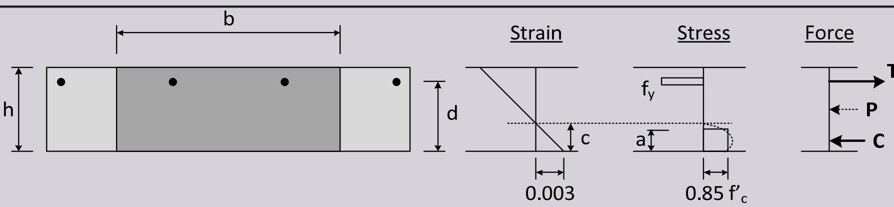
Overhang Capacity



$$T = sum \begin{cases} T_1 & (arises\ from\ moment, = C) \\ T_2 & (arises\ from\ axial\ force, = P) \end{cases}$$

$$a = \frac{A_s f_y - P}{0.85 f'_c b}$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) - P\left(\frac{h}{2} - \frac{a}{2}\right)$$

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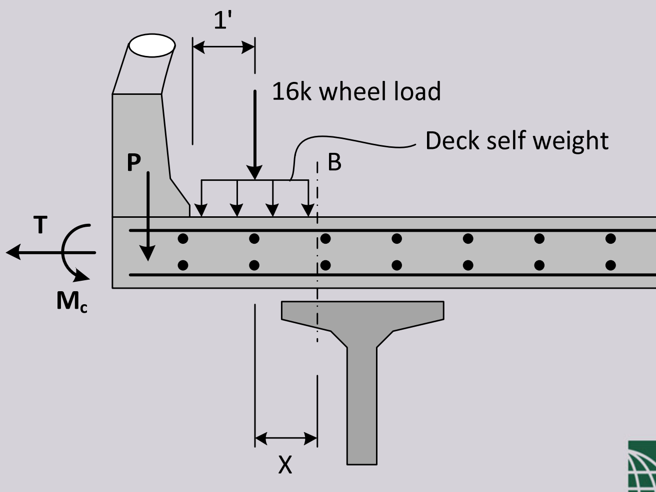
Case 3 – Section B



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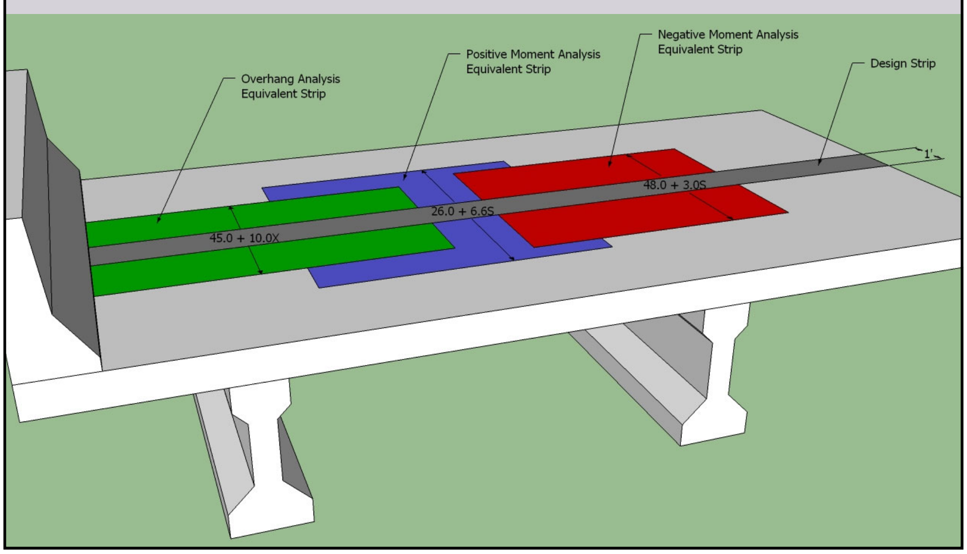
Overhang Equivalent Strip



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Simple Spans made Continuous

- Additional longitudinal reinforcement needed over interior piers to achieve ϕM_n in negative moment

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Key Points

- Decks are one-way slabs spanning between girders
- Four layers of reinforcement
  - » Primary negative reinforcement
  - » Primary positive reinforcement
  - » Distribution reinforcement
  - » Temp and Shrinkage reinforcement
- Equivalent strip width is the amount of concrete acting together to carry wheel load to girder
- Must consider horizontal collision force

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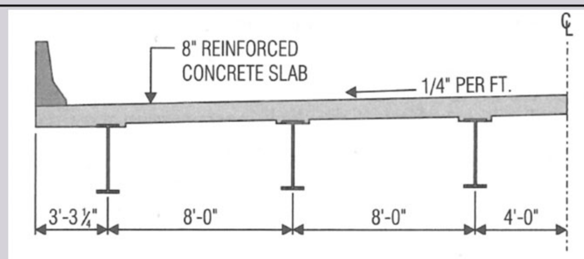
Example: Design of Reinforced Concrete Deck Slab



Given:
- Bridge to carry two traffic lanes.
- Concrete strength, $f_c{}'$ = 4.5 ksi.
- Grade 60 reinforcement, f_y= 60 ksi.
- Account for 25 psf future wearing surface.
- Assume supporting steel girders are W36 × 150.
- Deck has a 0.5 in. integrated wearing surface.
- Design lane = 12 ft. width.
- Deck is composite with beams.

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Example

Determine applicability of AASHTO Deck Design Tables (A 4 -1, p. 4-98).

a) IM and multiple presence factors considered.
b) More than three girders.
c) Distance between exterior girder (40 ft) > 14 ft

Compute moments due to dead loads.

Dead loads from slab and future wearing surface:

DC = ($thickness\ of\ slab$)($unit\ weight\ of\ concrete$) = (8 in)/(12 in)(0.15 kcf)
= 0.1 k/ft

DW = $future\ wearing\ surface$ = 0.025 k/ft
= 0.025 k/ft

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Example

Table 6.5.2—Approximate moments for nonprestressed continuous beams and one-way slabs

Moment	Location	Condition	M_u
Positive	End span	Discontinuous end integral with support	$w_u \ell_n^2/14$
		Discontinuous end unrestrained	$w_u \ell_n^2/11$
	Interior spans	All	$w_u \ell_n^2/16$
Negative[1]	Interior face of exterior support	Member built integrally with supporting spandrel beam	$w_u \ell_n^2/24$
		Member built integrally with supporting column	$w_u \ell_n^2/16$
	Exterior face of first interior support	Two spans	$w_u \ell_n^2/9$
		More than two spans	$w_u \ell_n^2/10$
	Face of other supports	All	$w_u \ell_n^2/11$
	Face of all supports satisfying (a) or (b)	(a) slabs with spans not exceeding 10 ft (b) beams where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of span	$w_u \ell_n^2/12$

[1]To calculate negative moments, ℓ_n shall be the average of the adjacent clear span lengths.

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Example

Positive bending moment/ft:

M_{DC}+ = (1/14)DC×S^2 = (1/14)(0.1 k/ft)(8 ft)^2
= 0.45 $kip-ft$

M_{DW}+ = (1/14)DW×S^2 = (1/14)(0.025 k/ft)(8 ft)^2
= 0.114 $kip-ft$

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Example

Equivalent strip widths (p. 4-24):

Positive moment: 26 + 6.6S = 26 + (6.6)(8) = 78.8 in.
Negative moment: 48 + 3.0S = 48 + (3.0)(8) = 72 in.

So, modified positive dead load moments/ft. (p. 4-23 commentary):

$M_{DC}+$ = 0.45(12*12/78.8) = 0.83 $kip{-}ft/ft$

Example

$M_{DW}+$ = 0.114(12*12/78.8) = 0.208 $kip{-}ft/ft$
= 0.208 $kip{-}ft/ft$

Negative bending moment/ft:

$M_{DC}-$ = $(0.10 DC \times S^2)$ = (0.10)(0.1 k/ft)(8 ft)^2
= 0.64 $kip{-}ft$
$M_{DW}-$ = $(0.10 DW \times S^2)$=(0.10)(0.025 k/ft)(8 ft)^2
= 0.16 $kip{-}ft$

Example

So, modified negative dead load moments/ft. (p. 4-23 commentary):

$M_{DC}-$ = 0.64(12*12/72) =1.28 $kip-ft/ft$

$M_{DW}-$ = 0.161(12*12/72) = 0.32 $kip-ft/ft$

Example

Compute live load bending moments:

AASHTO Table A4-1 (page 4.98) is used to obtain maximum approximate positive and negative HL-93 live load bending moments (Note: IM and multiple presence already factored in this table).

The deck span length is 8 ft. The top flange width for a W36 × 150 is 12 in, t_w = 0.625 in. (1/4 flange width = 3 in. is used for design negative moment location, p. 4-26).

M_L+ = 5.69 $kip-ft/ft$

M_L - = 5.65 $kip-ft/ft$

Table A4-1—Maximum Live Load Moments per Unit Width, kip-ft/ft

S		Positive Moment	Negative Moment						
			Distance from CL of Girder to Design Section for Negative Moment						
			0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4 ft	–0 in.	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4 ft	–3 in.	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4 ft	–6 in.	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4 ft	–9 in.	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5 ft	–0 in.	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5 ft	–3 in.	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5 ft	–6 in.	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5 ft	–9 in.	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6 ft	–0 in.	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6 ft	–3 in.	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6 ft	–6 in.	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6 ft	–9 in.	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7 ft	–0 in.	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7 ft	–3 in.	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7 ft	–6 in.	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7 ft	–9 in.	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8 ft	–0 in.	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8 ft	–3 in.	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8 ft	–6 in.	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8 ft	–9 in.	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9 ft	–0 in.	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9 ft	–3 in.	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9 ft	–6 in.	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9 ft	–9 in.	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10 ft	–0 in.	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10 ft	–3 in.	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10 ft	–6 in.	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10 ft	–9 in.	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11 ft	–0 in.	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11 ft	–3 in.	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11 ft	–6 in.	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11 ft	–9 in.	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12 ft	–0 in.	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12 ft	–3 in.	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12 ft	–6 in.	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12 ft	–9 in.	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13 ft	–0 in.	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13 ft	–3 in.	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13 ft	–6 in.	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13 ft	–9 in.	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14 ft	–0 in.	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45
14 ft	–3 in.	9.14	12.46	11.59	10.72	9.85	8.99	7.38	6.58
14 ft	–6 in.	9.25	12.67	11.81	10.94	10.08	9.21	7.57	6.72
14 ft	–9 in.	9.36	12.88	12.02	11.16	10.30	9.44	7.76	6.86
15 ft	–0 in.	9.47	13.09	12.23	11.37	10.51	9.65	7.94	7.02

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Example

So, modified live load moments/ft. (p. 4-23 commentary):

M_L+ = 5.69(12*12/78.8) = 10.4 $kip\text{–}ft/ft$
M_L– = 5.65(12*12/72) = 11.3 $kip\text{–}ft/ft$

Compute total factored bending moments (Strength I).
M_f+ = 1.25M_{DC}+ + 1.5M_{DW}+ +1.75M_L+
= (1.25)(0.83 $kip\text{–}ft$) + (1.5)(0.208 $kipt\text{–}ft$) + (1.75)(10.4 $kip\text{–}ft$) = 19.55 $kip\text{–}ft/ft$

M_f– = 1.25M_{DC}–+1.5M_{DW}–+1.75M_L–
= (1.25)(1.28 $kip\text{–}ft$) + (1.5)(0.32 $kip\text{–}ft$) + (1.75)(11.3 $kip\text{–}ft$) = 21.85 $kip\text{–}ft$/ft

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Example

Compute effective depth of slab:
· Slab depth = 8 in
· Sacrificial surface = 0.5 in
· Concrete cover = 2.5 in top and 1.0 in bottom (p. 5-165)
· #5 reinforcing steel bar diameter = 0.625 in

The effective depths of the slab:
d_+= 8.0 − 0.5 − 1.0− 0.625/2 = 6.19 in
d_-= 8.0 −2.5 − 0.625/2 = 5.19 in

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Example

Positive bending moment design (use any method of preference):

$R_n = M_u/\emptyset bd^2$ = 19.55(12000)/1.0(12)6.192 = 510 psi

From design aid,
ρ = 0.0092, and A_s = 0.0092(12)(6.19) = 0.68 in^2/ft
Now, using #6 rebars:
s+ = 0.44*12/0.68 = 7.76 in, use 7 in. o.c.

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Example

Negative bending moment design:

$R_n = M_u/\emptyset bd^2$ = 21.85(12000)/1.0(12)5.19^2 = 799 psi

From design aid,

ρ = 0.0149, and A_s = 0.0149(12)(5.19) = 0.93 in^2/ft.

Now, using #6 rebars:

s_{-}= 0.44*12/0.93 = 5.67 in, use 5 in. o.c.

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Example

Check maximum spacing for tension steel (p. 5-168):

s_{max}= 1.5∗8 or 18 in, OK

Distribution reinforcement in bottom of slab (p. 9-12):

The effective span length:

S = 8.0 ft − 0.05ft = 7.95 ft

D = 220/$\sqrt{S}$ = 2 20/√7.95 = 78% > 67%, so use 67%

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Example

Distribution reinforcement = $(A_s)(67\%)$ = $(0.68\ in^2/ft)(0.67)$
= $0.46\ in^2/ft$

So, use no. 6 bars at 11 in. o. c.

T&S reinforcement at the top of slab (p. 5-170):

$A_s \geq 1.30(12)(8)/2(12 + 8)60 = 0.052 < 0.11$ minimum

So, use $A_s = 0.11\ in^2$

Use #3 @ 12 in. o.c., satisfies 3h or 18 in. max spacing.

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Segmental Deck Design

- Design should be checked for construction load combinations, such as segment lifting, construction equipment, and segment stacking.
- It is standard practice to select a minimum top deck thickness of 8 in, although AASHTO-PCI-ASBI Standard Sections Committee recommends a minimum thickness of 9 in.
- It is recommended that for all posttensioned box girders the top deck be transversely posttensioned, even for short overhangs since it improves long-term durability and results in low life cycle cost.
- When a static concentrated load is applied on a deck, the deck will deflect transversely as well as longitudinally, similar to two-way slabs.

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Transverse Analysis

- ✓ To evaluate transverse bending moments from both permanent and live loads.
- ✓ Results are used to design the reinforcing and post-tensioning of the cantilever wings, top slab, webs, and bottom slab.
- ✓ An accepted approach for the transverse analysis of a concrete box girder superstructure is a simplified 2D approach with sufficient consideration for longitudinal load distribution.

One Foot Section Segmental Bridge
for Transverse Analysis

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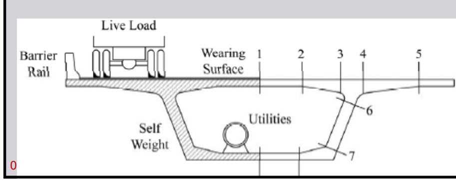
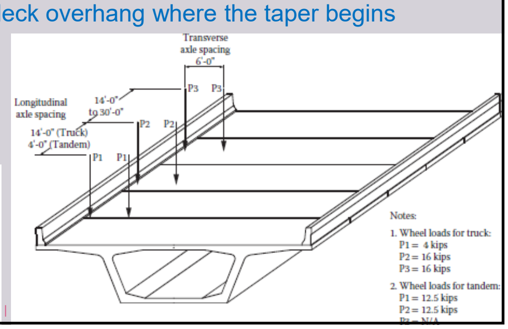
Transverse Analysis

- ➤ Live load is strategically placed to produce worst condition. Some common points where stresses are checked are:
  - ✓ Maximum negative bending moment at the root of deck overhang
  - ✓ Maximum positive and negative bending moments at the center line between two webs
  - ✓ Maximum negative bending moment in the top deck at the interior face of the webs
  - ✓ Maximum negative and positive bending moments in the webs and bottom slab
  - ✓ Maximum negative moment in the deck overhang where the taper begins





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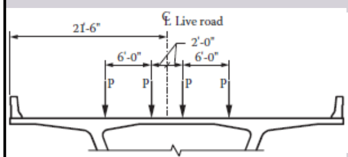
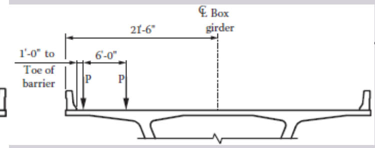
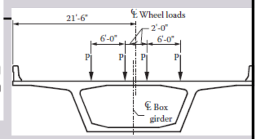
LRFD Live Load Transverse Configurations



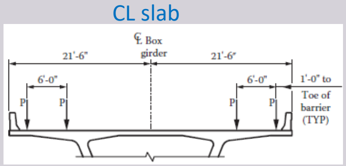
Live load configuration for maximum positive moment at CL slab



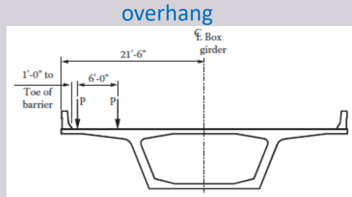
Live load configuration for maximum negative moment at overhang



Live load configuration for maximum negative moment in top slab at inside face of web



Live load configuration for maximum negative moment at CL slab



Live load configuration for maximum negative moment in bottom slab at inside face of web

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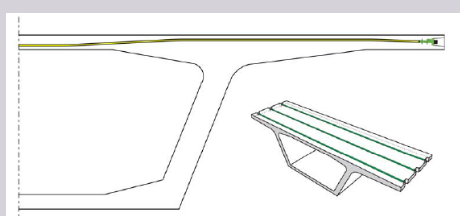
Transverse Post-tensioning

- The cantilever wings and the top slab are typically prestressed transversely with post-tensioning to offset tensile stresses resulting from permanent and live loads.
- Narrow precast box girders with widths of 16 ft. or less, often used for single track transit systems, may not greatly benefit from transverse post-tensioning.



Typical Transverse Tendon Layout

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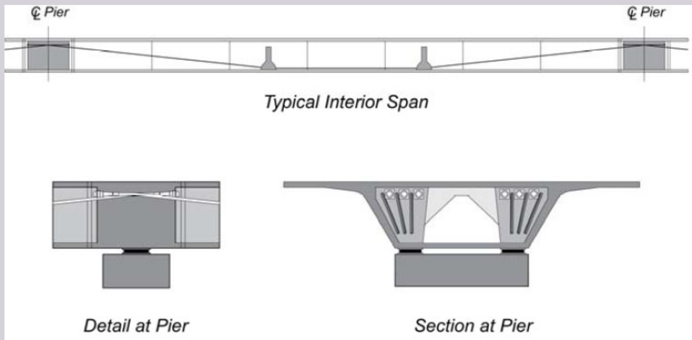
Longitudinal Design

- The longitudinal design depends on the erection method.



Example Tendon layout for Span-By-Span Segmental Bridge

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Video Clips and Software

- Segmental Bridge Construction
- Precast yard
- Software

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Questions?

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