INDOT
Structures Conference

Major Changes to
Indiana Design Manual Chapter 61
Bridge Decks

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American Structurepoint, Inc.
7260 Shadeland Station
Indianapolis, IN 46256
(317) 547-5580

Chapter 61 Changes

- Extensive background section has been removed.
- Chapter now begins with strip method of design.
- Patch method has been deleted. Don’t know if anyone had ever designed a slab this way.

Chapter 61 Changes

- Empirical method has been deleted.
- The design examples for the deck overhang and transverse edge beam have been removed.
- Numerous figures that relate to the above changes have been removed.

Chapter 61 Changes

- 61-1.02 #1 (was 61-2.02 #1) It is not necessary to have the same size bar or same spacing for top and bottom reinforcing steel in the primary direction. For constructability, the top and bottom bars should be the same size.
- We removed the last statement, it seemed contradictory.

Chapter 61 Changes

- 61-2.01 #2 (was 61-4.01 #2) The primary reinforcing steel should be at a maximum spacing of 8 inches. Requirement is for primary, distribution and temperature steel. This is not enforced today, but it will be coming.

Chapter 61 Changes

- Permanent metal deck forms language has been rewritten as the norm.
- Overhang criteria of 0.85 x Beam Depth is now *suggested*. The overhangs require design and now with construction loadings according to Design Memo 10-18.

Bridge Deck Design Example

The bridge deck design example is presented as a straight forward typical bridge deck design with only the necessary design checks. The simplifications are based on the following.

1. The deck is designed using the table in appendix A4 of the AASHTO LRFD Bridge Design Specifications.

2. The Deck Slab Design Table has a few requirements such as the deck must have at least 3 girders and a width of 14 feet between the exterior girders.

   The overhang must be at least 21 inches but not more than 6 feet measured from the centerline of the exterior girder.

3. Remember that Ø = 0.9 for the strength limit state (S 5.4.4.2)
and Ø = 1.0 for the extreme event limit state (S1.3.2.1)

4. n = 1 for bridge decks

The writer of the design manual spent pages arguing the fact that this method of deck design was too conservative, he demonstrated it was 15 to 20 percent more conservative than his chosen method and since an additional 5% loading is not a satisfactory method to improve the structures performance it is recommended n=1 for bridge decks.

5. This example excludes the 2nd and 3rd design cases.

Design Case 2: The vertical collision force (SA13.4.1) never controls for a concrete parapet.

Design Case 3: The DL + LL (SA13.4.1) this case only controls for narrow flange beams with a beam spacing greater than 12 feet and wide flange beams with a beam spacing greater than 14 feet.

6. The exposure factor used in the crack control is 1.0 as recommended by Professor Frosch.

The full deck thickness is used in the calculation. (this is not a structural calculation).

7. The maximum spacing used for the temperature and distribution steel is 8 inches.

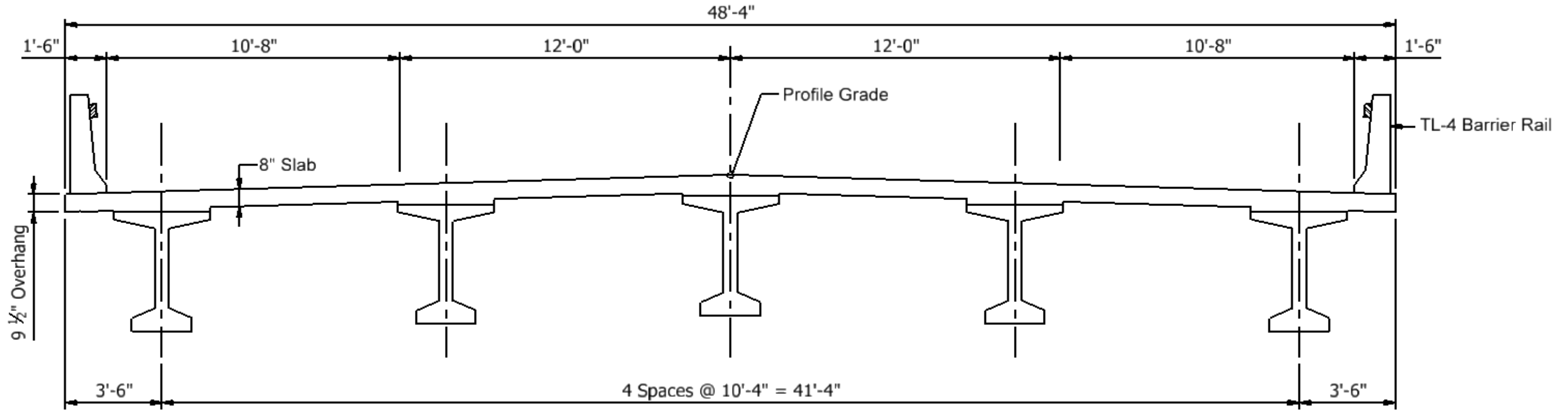
8. The additional deck steel in the deck does not use a hook.

Bridge Deck Design Example

In this design example, the deck will be designed for dead and live loads at the strength limit state. The design will check the deck for vehicular collision with the railing system at the extreme event state.

This design example will follow the approximate method of deck design (S4.6.2-1).

This method is typically referred to as the equivalent strip method.



Design Data:

Girder spacing	10'-4"
Top cover	2½" (includes ½" wearing surface) (S5.12.3)
Steel yield strength	60 ksi
Concrete compressive strength	4 ksi
Deck Weight	150 pcf.
Future wearing surface	35 psf.
Deck thickness	8 in.
Overhang thickness	9½ in.

Load Factors

Deck and bridge rail 1.25

Future wearing surface 1.5

M = Dead load positive or negative moment in the deck for a unit width strip (K-ft/ft).

$M = w\, l^2 / c$

W = Dead load per unit area of the deck

l = girder spacing

c = constant, taken as 10

Self weight of deck = 8(150)/12 = 100 psf

Unfactored self weight moment = $(100/1000)\ (10.33)^2\ /\ 10 = 1.07$ k-ft/ft

Future wearing surface 35 lb/ft^2

Unfactored FWS = $(35/1000)\ (10.33)^2\ /10 = 0.37$ k-ft/ft

Distance from the center of the girder to the design section for negative moment (Table A4-1) (S4.6.2.1.6)

For precast I - shaped and T - shaped concrete beams, the distance from the centerline of girder to the design section for negative moment in the deck should be taken equal to one-third of the flange width from the centerline of the support, but not to exceed 15 in.

$\frac{1}{3}(48) = 16''$ use 15"

Interpolated Negative moment	4.58 kip ft/ft
Interpolated Positive moment	7.08 kip ft/ft

Design For Positive Moment In The Deck

Factored Loads:

Live Load: Maximum factored positive moment per unit width

= 1.75(7.08) = 12.39 k-ft/ft

Deck weight: 1.25 (1.07) = 1.34 k-ft/ft

FWS: 1.5 (0.37) = 0.56 k-ft/ft

Dead load + live load design factored positive moment (Strength 1 limit State)

MDL+ LL = 12.39 + 1.34 + 0.56

= 14.29 k-ft/ft

For the positive moment section:

de = effective depth from the compressive fiber to the centroid of the tensile force in the tensile reinforcement (in.)

= total thickness - bottom cover - ½ bar diameter - integral wearing surface

= 8 - 1- ½(0.625) - ½

= 6.19"

ρ = 0.85 (f'c/fy)[1.0- $\sqrt{1.0 - 2k'/0.85f'c}$]

= 0.00738

K' = Mu / (Øbd²)

= 14.29/[0.9(1)(6.19)²]

= 0.414 k/in²

As = ρ de = (0.00738)(6.19) = 0.0457 in²/in

Area of #5 bar = 0.31 in²

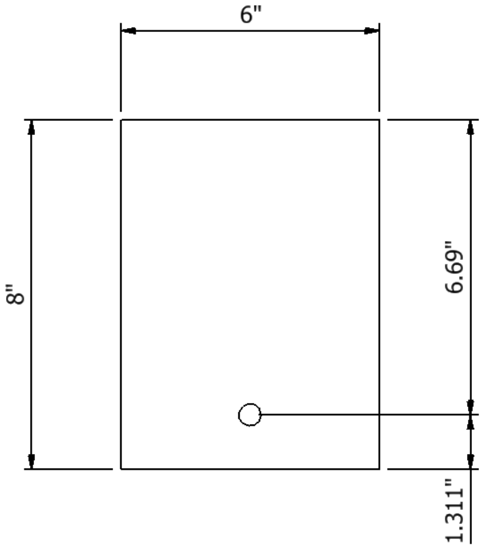
0.31/0.04857 = 6.78"

Use #5 bar spaced at 6"

(S5,7,3,4)

$s \leq 700\,\gamma e / \beta s\, fss - 2dc$

$\beta s = 1 + dc / 0.7\,(h - dc)$



γe = exposure factor, let $\gamma e = 1.0$

dc = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest there to (in.)

fss = tensile stress in steel reinforcement at the service limit state (ksi)

h = overall thickness

dt = distance from extreme compression fiber to the centroid of extreme tension element (in.)

$\rho = As / bd = 0.31 / (6)(6.6875) = 0.0077$

$Ec = 3640$ ksi

$Es = 29000$ ksi

$n = Es/Ec = 8$

Unfactored Loads:

Deck	1.07
FWS	0.37
Live Load	<u>7.08</u>
	8.52 k-in/in

$\beta s = 1.28$

$\rho n = (.0077)(8) = .0616$

$k = \sqrt{2pn + (pn)^2} - pn = 0.295$

$j = 1 - k/3 = 0.902$

Service moment

$(8.52)(6) = 51.12$

$fss = Ms / As\ j\ dt = 27.33$ ksi

$S = 17.4''$

Design For Negative Moment At Interior Girders

Live Load:	1.75(4.58) = 8.0 k-ft/ft
Dead Load:	1.25(1.07) = 1.34 k-ft/ft
FWS:	1.5(0.37) = 0.56 k-ft/ft

Dead load + Live load design factored negative moment

= 1.34 + 0.56 + 8.0 = 9.9 k-ft/ft

d = distance from compression force to centroid of tension reinforcement (in.)

= total thickness - top cover - ½ bar diameter

For a #5 bar

$d = 8 - 2\frac{1}{2} - \frac{1}{2}(0.625) = 5.19$ in.

$k' = M_u / \varnothing b d^2 = 9.9/[0.9(1)(5.19)^2] = 0.408$ k/in.^{2}

$\rho = 0.85(f'c/fy)[1.0 - \sqrt{1.0 - 2k'/0.85f'c}\,] = 0.00726$

$A_s = \rho\, d_e = (0.00726)(5.19) = 0.03768\ in^2/in$

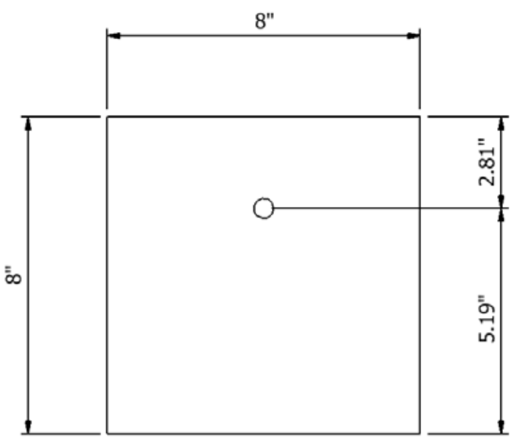
$0.31/0.03768 = 8.23"$ **use 8"**

Area of #5 bar 0.31 in^2

(S5,7,3,4)

$s \leq 700\ \gamma_e / \beta_s f_{ss} - 2d_c$

$\beta_s = 1 + d_c / 0.7 (h - d_c)$



γ_e = exposure factor, let $\gamma_e = 1.0$

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest there to (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)

h = overall thickness

d_t = distance from extreme compression fiber to the centroid of extreme tension element (in.)

$\rho = As / bd = 0.31 / (8)(5.19) = 0.00747$

$Ec = 3640$ ksi

$Es = 29000$ ksi

$n = Es/Ec = 8$

$\rho n = (.00747)(8) = .06$

$$k = \sqrt{2pn + (pn)^2} - pn = 0.292$$

$j = 1 - k/3 = 0.903$

$Ms = 48.16$

$fss = 33.15$

$ßs = 1.773$

$S = 6.3"$

use 6"

Deck Overhang Design (A13.3.2.1-2) 61-5.02(02)

Rw = 1.25 (54) = 67.5 kips

$$Lc = Lt/2 + \sqrt{(L_t/2)^2 + \frac{8H(M_b + M_w)}{M_c}}$$

T = Rw / (Lc + 2H) = 4.56 k/ft

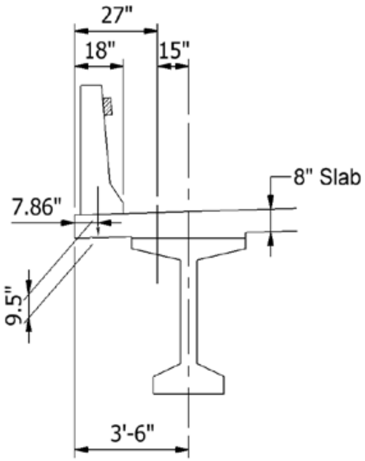
H = total height of wall Lc = 9 ft.

Lt = 3.5 Mb = 0 Mw = 43.6 k-ft

with Rw equal to 67.5 kip solve for Mc

$$Rw = \left(\frac{2}{2L_c - L_t}\right)(8Mb + 8Mw + Mc\ Lc^2/H)$$

Mc = 16.6 k-ft/ft



Load		Arm	Factor	Moment
Rail	0.388	1.6	1.25	0.776
Deck	(.118)(2.25)	1.125	1.25	0.373
FWS	(.035)(0.75)	0.375	1.5	0.015

Mn = 1.25Mdc + 1.5Mdw + Mc
= 17.76

The required flexural resistance with the axial tension included equals:

17.76 + 4.65 = 22.41 k-ft/ft

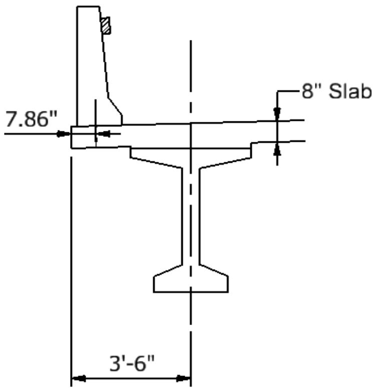
dc = 9.5 -2.5 - 0.3125 = 6.6875"

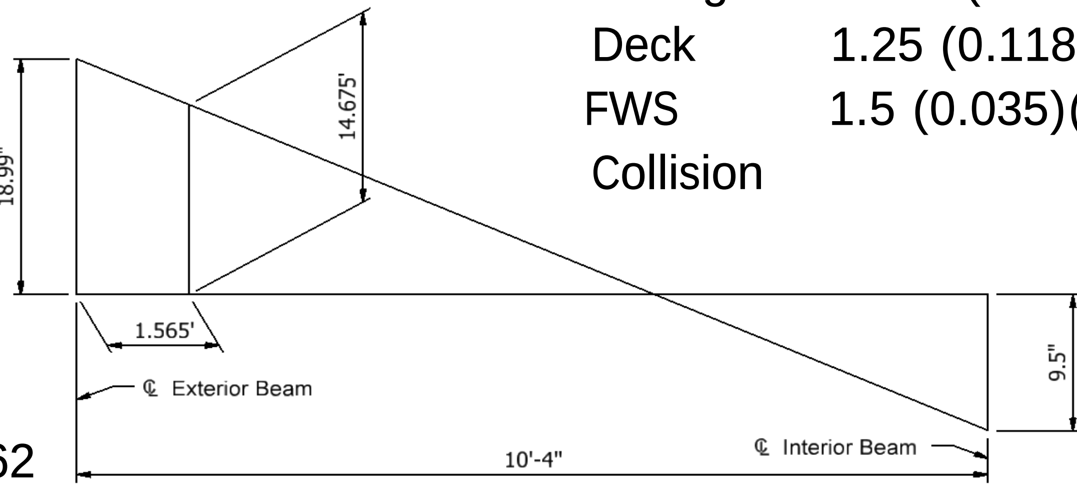
As = ρ de = 0.0608 in^2/in

0.51/0.0682 = #5bar and #4bar @ 8.4"

<u>use #4 bar between the #5 bars</u>

Simplified Cut-Off Point





Railing	1.25 (0.388)(2.85)	= 1.38
Deck	1.25 (0.118)(3.5)(1.75)	= 0.90
FWS	1.5 (0.035)(2)(1)	= 0.105
Collision		= 16.6
		18.99

12/6 x 0.31 = 0.62

$$a = \frac{A_s f_y}{0.85(f'c)(12)} = 0.912$$

$$\varnothing Mn = \frac{\phi A_s f_y}{b}\,[d - a/2] = 14.67$$

d = 8 - 2.5 - 0.3125 = 5.19

(S5.11.1.2.1) effective depth of member
15 x bar dia
1/20 of the clear span

15 x 5/8 = 9.375 controls **use 6'**

Longitudinal Reinforcement

(S9.7.3.2) Bottom Distribution Reinforcement

S = distance between flange tips + distance from flange tip to face of web

= 6'-4" + 1'-8½" = 8'-½"

$\frac{220}{\sqrt{S}}$ = 77.6% use 67%

$\frac{220}{\sqrt{S}} \leq$ 67% #5 bar @ 6"

12/6 x 0.31 = 0.62 in^2/ft

0.67 (0.62) = 0.4154 in^2/ft

0.31/ 0.4154 = 0.747' or 8.96" **use 8"**

Max Spacing 8"

Temperature Steel

(S5.10.8-1) $Ag \geq \frac{1.30\ bh}{2(b+h)fy}$ = .085 in^2/ft

(S5.10.8-2) $0.11 \leq Ag \leq 0.60$

MAX SPACING 8" #4 bar @ 8" Ag = 0.3 in^2/ft

Practice Pointers

Reinforcing Steel

- Recommended that designers use standard lengths bars in the deck and railing as much as possible.
- Standard lengths are 30 feet and 40 feet. (and 60 feet, but anything over 48 feet requires a special permit)
- Advantage is the bars can be epoxy coated and shipped directly to job site without any further cutting or fabrication.
- 40’ standard bars can be bundled in 9000# bundles for shipping. Cut bars have a maximum bundle size of 4500#.

Practice Pointers

Reinforcing Steel

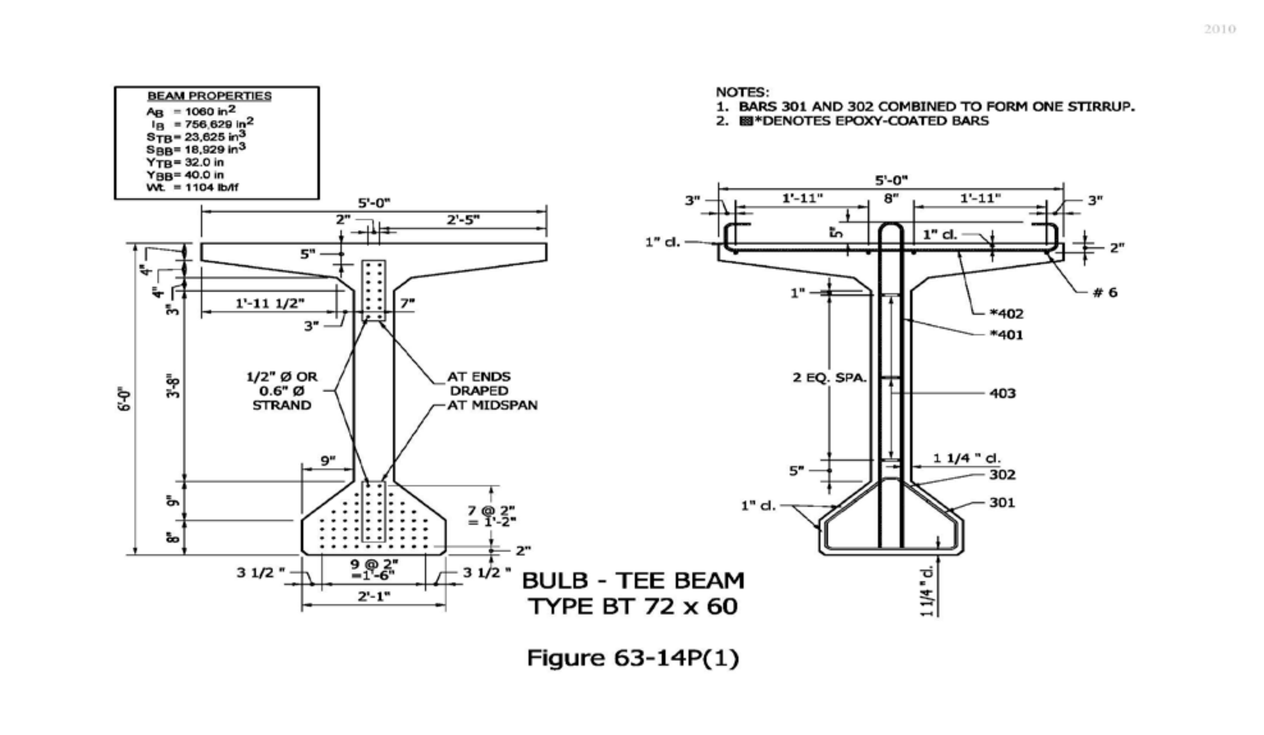
- The transverse steel in the deck overhangs usually requires additional steel. Recommend using a standard bar and splice a larger bar for the additional steel in the overhang.
- Contractors present at the meeting prefer using fewer bars in the overhang.
- Contractors recommend using 30’ bars for temperature steel because longer bars cause handing problems.
- Smaller county projects may only have an excavator on site for moving bundles.

Practice Pointers

Screeds

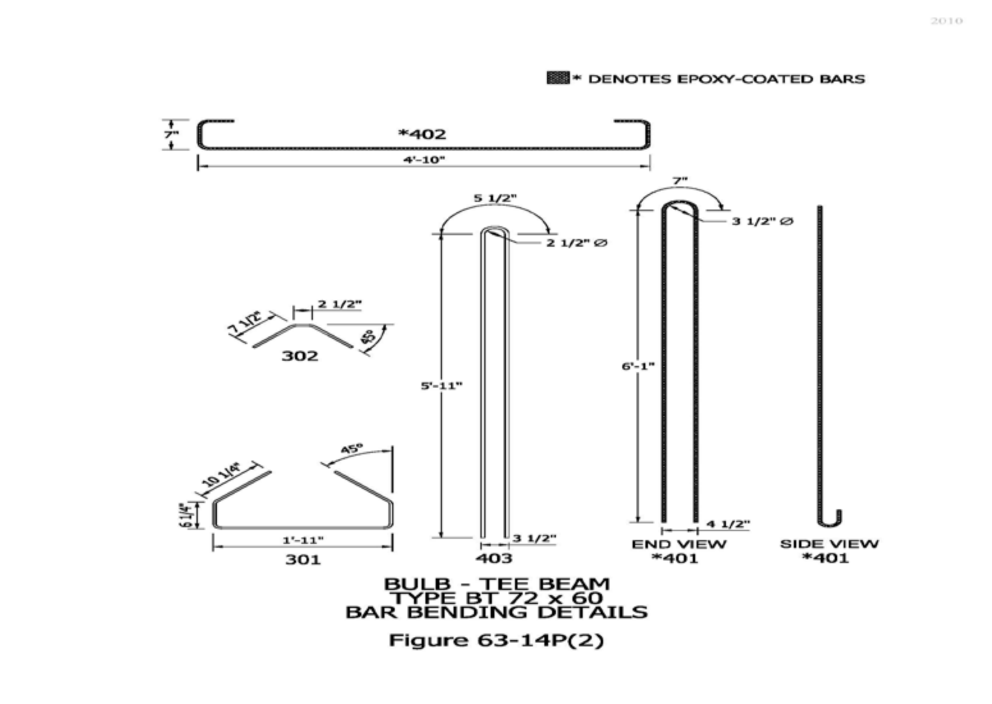
- Revised Chapter 61 recommends rounding screed elevation to the nearest 0.005 feet.
- Contractors recommend (prefer) to have a screed shot on each edge of a bulb tee top flange.
- Eliminates the need for leveling shots from center of beam.

Practice Pointers



Typical Reinforcing for INDOT Bulb Tee Beam

Practice Pointers



Typical Reinforcing for INDOT Bulb Tee Beam

Practice Pointers

Precast Concrete Beam Reinforcing

- The 401 bar is the basic vertical shear steel reinforcement bar.
- A few years back, this bar was given hooks.
- INDOT is the only state DOT with hooks.
- Recommended that we eliminate the hooks, but this thought is meeting resistance.

Practice Pointers

Precast Concrete Beam Reinforcing

- The 402 bar is the horizontal shear steel. For simplicity this bar has usually been spaced with the 401 bar for vertical shear.
- Vertical shear requirements have increased significantly for LRFD beam designs.
- Recommended that we space the 402 bar based on need and not match the 401 bar spacing.

Practice Pointers

Precast Concrete Beam Reinforcing

- The 403 bar is placed horizontally at the ends of the beams. Intended to limit cracking.
- Beam cracks tend to develop due to a beam notch or excessive release stresses.
- Draped strands, additional top strand(s) and elimination of beam notch would be more in line with industry standards.
- Manufacturers recommend elimination of the bar because it tends to form a dam and cause pouring problems.

Practice Pointers

Precast Concrete Beam Reinforcing

- The 301 bar is placed at the end of the beam to prevent developmental bursting.
- Some designers are placing these bars throughout the length of the beam.
- These bars are not needed full length
- We recommend Design Manual details be revise to reflect clearer direction.

Practice Pointers

Precast Concrete Beam Reinforcing

- The beam manufacturers recommend that designers use a standard distance between draped strand deflection points.
- Typically the distance between points is 0.3L and 0.4L, where L is the length of the beam.
- Distance should be consistent for a group of beams if there are minor variations in beam length.
- Round distance to the nearest whole foot.

Practice Pointers

Recommended Practices

Maximum Slope of Draped Strands for Given Size of Strand

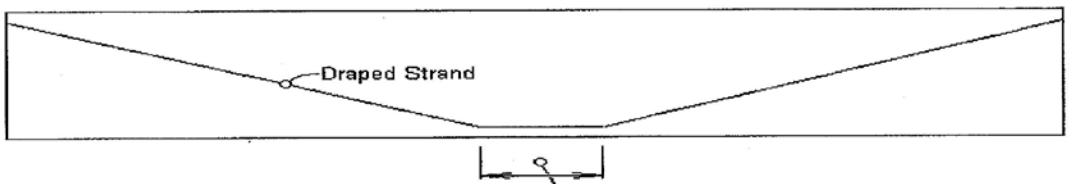
The use of these maximum slopes for draped stands will maintain an **maximum uplift per strand** value within the allowable 4,000 lbs** force per the Strand Restraining Device (holddown) manufactures' **Safe Working Load** recommendations.

Strand Type	Area in^2	Maximum * Rise / Run
3/8" ø 270K ***	0.085	0.225
7/16" ø 270K ***	0.115	0.165
1/2" ø 270K	0.153	0.123
1/2" ø 270K - Oversize	0.167	0.113
9/16" ø 270K ***	0.192	0.098
0.6" ø 270K	0.217	0.086

* Initial tension at 75% of ultimate

** Friction losses included

*** Not recommended for standard bridge beams.



Suggestions

Some popular beam design softwares defaults this distance at 0.4 L of the beam. **Prestress Services** suggests to the designer to set this distance at:

Beams <= 50 feet -- 5 feet (2'-6 each way of the centerline of the beam)
Beams > 50 feet -- 10 feet (5'-0 each way of the centerline of the beam)

Prestress Services has found that most beams with a length-to-depth ratio of 20 or less, generally, may be designed with all parallel strand <u>and</u> no debonding

Recommended Draping Criteria

Comments