All IDOT Design Guides have been updated to reflect the release of the 2017 AASHTO LRFD Bridge Design Specification, 8th Edition. The following is a summary of the major changes that have been incorporated into the Deck Design Guide.

- Many references to Section 5 of AASHTO have been updated to reflect the reorganization of the section.

- Various concrete equations were updated in AASHTO to include a concrete density factor, \( \lambda \). For normal weight concrete \( \lambda = 1 \) and, therefore, has been omitted from the equations in this guide for simplicity.

- The equation for the concrete modulus of elasticity, \( E_c \), has been modified.

- Per ABD Memo 15.8 the skew reduction factor, \( r \), shall be applied to the live load longitudinal force effect.

- The definition of \( f_s \) has been revised to reflect the current code language.

- The definition of \( \alpha_1 \) was added.

- The modular ratio will now be taken as an exact value as opposed to assuming a value of 9.

- Limits of Reinforcement have been changed to Minimum Reinforcement and all of the sections relating to Maximum Reinforcement have been removed.

- Values used in the example problem have been updated to reflect current standards (i.e. \( f'_c = 4 \) ksi, \( w_c = 0.145 \) kcf for calculation of \( E_c \), etc.)
3.2.1 LRFD Deck Design

A standard deck is defined as a deck slab on longitudinal beams with main reinforcement placed perpendicular to traffic.

As outlined in Article 9.6.1, the AASHTO LRFD Bridge Design Specifications, 8th Ed. permits three methods or procedures for designing bridge decks with primary reinforcement perpendicular to the main bridge beams. These are: (a) Approximate Elastic or “Strip” Method (4.6.2.1); (b) Empirical Design (9.7.2); and (c) Refined Analysis (4.6.3.2). The LRFD Deck Design Chart in Section 3.2.1 of the Bridge Manual was developed using the Strip or Approximate Elastic Method. Refined Analysis utilizes finite elements, which is unnecessary for standard deck design. Empirical Design utilizes prescriptive detailing as opposed to structural design, and is not advocated by IDOT.

In the Approximate Elastic Method, the deck is designed for Flexural Resistance (5.6.3.2) and Control of Cracking (5.6.7). Limits of Reinforcement are also checked, but do not typically control in a standard deck design.

Shear design is not required for deck slabs (C4.6.2.1.6). Fatigue and Fracture design is also not required (9.5.3).

In a standard deck, three components are designed. Positive moment (bottom of slab transverse) reinforcement and negative moment (top of slab transverse) reinforcement are designed for the Approximate Elastic Method. Additional negative moment reinforcement for deck overhangs, designed for Extreme Event II (vehicular collision) Loading, is added as per the details provided in Section 3.2.4 of the Bridge Manual. This additional reinforcement only needs to be designed if the deck overhang exceeds 3 ft. 8 in. in length. The concrete barriers, in conjunction with the deck overhang designs, are rated for Crash Test Level TL-4 and TL-5, respectively, which is adequate for most situations.

Longitudinal reinforcement is not designed. The top longitudinal reinforcement need only satisfy Shrinkage and Temperature Requirements (5.10.6), where #5 bars at 12 in. centers are adequate. The bottom longitudinal reinforcement area is a percentage of the bottom transverse
reinforcement (9.7.3.2). The percentage is 67% for all bridges with beam spacings within the limits of the standard deck design charts.

Additional longitudinal reinforcement is required for continuous span structures over the piers. See Sections 3.2.2 and 3.2.4 of the Bridge Manual for more information.

Reinforcement shall be developed to satisfy Section 5.10.8 of the LRFD Code. Extending the negative moment reinforcement to the end of slab and the positive moment reinforcement to one foot from the end of slab satisfies these requirements.

LRFD Deck Slab Design Procedure, Equations, and Outline

Design Stresses

\[ f_c' = 4.0 \text{ ksi} \]
\[ f_y = 60 \text{ ksi} \]

Design Thickness

The IDOT standard slab thickness is 8 in. for all girder spacings less than or equal to 9 ft. - 6 in. For girder spacings exceeding 9 ft. – 6 in., the standard design charts are not applicable.

Determine Maximum Factored Loading

When designing deck slabs, two load combinations are used:

Strength I load combination, used in Flexural Resistance, is defined as:

\[ M_{\text{STRENGTH I}} = \gamma_p DC + \gamma_p DW + 1.75(LL + IM) \]  
(Table 3.4.1-1)

Where \( \gamma_p \) is equal to 1.25 (max.) for DC and 1.5 (max.) for DW.

Service I load combination, used in Control of Cracking, is defined as:
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\[ M_{\text{SERVICE}} = 1.0(DC + DW + LL + IM) \]  
(Table 3.4.1-1)

The load abbreviations are defined as follows:

- **DC** = dead load of structural components (DC1) and non-structural attachments (DC2). Standard deck slabs are not designed for DC2 loading.
- **DW** = dead load of future wearing surface, taken as 50 psf for standard IDOT bridge deck designs
- **LL** = vehicular live load
- **IM** = dynamic load allowance (impact)
- **LL** = vehicular live load

Dead load (DC1 and DW) design moments are computed as \( wL^2/10 \). L is defined as the center-to-center beam spacing (4.6.2.1.6) for positive moment calculation. For negative moment, L is taken as that defined in Bridge Manual Figure 3.2.1-2.

Standard parapet, sidewalk, and railing loads are considered DC2 loading and are not used in the main reinforcement design. Bridges with large additional DC2 loads may require a non-standard deck design.

Live loads are taken from AASHTO LRFD Table A4-1. This table gives the Live Load Moment per ft. width for a given beam spacing. These values are already corrected for multiple presence factors and impact loading. Note that Bridge Manual Figure 3.2.1-2 defines span lengths for negative moment regions differently than positive moment regions. These span lengths fall within the limitations of AASHTO LRFD Section 4.6.2.1.6.

As per Article 3.6.1.3.3 of the AASHTO LRFD Bridge Design Specifications, when designing bridge decks, loads need not be amplified for centrifugal and superelevation effects.

All factored loads are then multiplied by the load modifier \( \eta_i \), defined as:

\[ \eta_i = \eta_D \eta_R \eta_i \geq 0.95 \]  
(1.3.2.1-2)
Where:

\[ \eta_D = \text{ductility factor, taken as 1.00 for conventional designs} \]
\[ \eta_R = \text{redundancy factor, taken as 1.00 for conventional levels of redundancy} \]
\[ \eta_I = \text{importance factor, taken as 1.00 for typical bridges} \]

For most bridges, \( \eta_I = (1.00)(1.00)(1.00) = 1.00 \)

**Check Flexural Resistance**  
(5.6.3.2)

The factored resistance, \( M_r (\text{k-in.}) \), is taken as:

\[
M_r = \phi M_n = \phi \left[ A_s f_s \left( d_s - \frac{a}{2} \right) \right] \geq M_{\text{STRENGTH1}} \quad \text{(Eqs. 5.6.3.2.1-1 & 5.6.3.2.2-1)}
\]

Where:

\( \phi = \text{Assumed to be 0.9, then checked using the procedure found in Article 5.5.4.2. In this procedure, the reinforcement strain, } \varepsilon_t, \text{ is calculated, and } \phi \text{ is dependent upon this strain. } \varepsilon_t \text{ is calculated assuming similar triangles and a concrete strain of 0.003.} \)

\[
\varepsilon_t = \frac{0.003(d_t - c)}{c} \quad \text{(C5.6.2.1)}
\]

- If \( \varepsilon_t < 0.002, \phi = 0.75 \)
- If \( 0.002 < \varepsilon_t < 0.005, \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{II} - \varepsilon_{cl})} \)
- If \( \varepsilon_t > 0.005, \phi = 0.9 \)

Where \( \varepsilon_{cl} \) is taken as 0.002 and \( \varepsilon_{II} \) is taken as 0.005, as stated in Article 5.6.2.1.

\( a = \text{depth of equivalent stress block (in.), taken as } a = \beta_1 c \)
\[
c = \frac{A_s f_s}{\alpha_1 \beta_1 f'_c} \text{ (in.)} \quad \text{(Eq. 5.6.3.1.1-4)}
\]
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3.2.1 - LRFD Deck Design

\[ A_s = \text{area of tension reinforcement in strip (in.}^2\text{)} \]
\[ b = \text{width of design strip (in.)} \]
\[ d_s = \text{distance from extreme compression fiber to centroid of tensile reinforcement (in.)} \]
\[ f_s = \text{stress in the mild steel tension reinforcement as specified at nominal flexural resistance (ksi). As specified in Article 5.6.2.1, if } c / d_s < 0.003 / (0.003 + \varepsilon_{cl}), \text{ then } f_y \text{ may used in lieu of exact computation of } f_s. \text{ For 60 ksi reinforcement, } \varepsilon_{cl} \text{ is taken as 0.002, making the ratio } 0.003 / (0.003 + \varepsilon_{cl}) \text{ equal to 0.6. Typically in design, } f_s \text{ is assumed to be equal to } f_y, \text{ then the assumption is checked.} \]
\[ f_c = \text{specified compressive strength of concrete (ksi)} \]
\[ \alpha_1 = 0.85 \text{ for concrete with strength less than 10 ksi (5.6.2.2)} \]
\[ \beta_1 = \text{stress block factor specified in Article 5.6.2.2} \]

\[ \therefore M_r = \phi M_n = \phi \left[ A_s f_s \left( d_s - \frac{1}{2} \frac{A_s f_s}{0.85 f_c b} \right) \right] \]

Check Control of Cracking (5.6.7)

The spacing of reinforcement, s (in.), in the layer closest to the tension face shall satisfy the following:

\[ s \leq \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c \]  
(Eq. 5.6.7-1)

Where:

\[ \beta_s = 1 + \frac{d_s}{0.7(h - d_c)} \]  
(Eq. 5.6.7-2)

\[ d_c = \text{thickness of concrete cover from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)} \]
\[ h = \text{slab depth (in.)} \]
\[ f_{ss} = \text{stress in mild steel tension reinforcement at service load condition, not to exceed } 0.6f_y \]
\[
\begin{align*}
\text{j} &= 1 - \frac{k}{3} \\
k &= \sqrt{(\rho n)^2 + 2\rho n - \rho n} \\
\rho &= \frac{A_s}{bd_s} \\
n &= \frac{E_s}{E_c}
\end{align*}
\]

\[
E_s = 29000 \text{ ksi} \quad \text{(6.4.1)}
\]

\[
E_c = 120000K_1w_c^{t_c^{0.33}} \quad \text{(Eq. 5.4.2.4-1)}
\]

\[
K_1 = 1.0 \text{ for normal-weight concrete} \\
w_c = 0.145 \text{ kcf} \quad \text{(Table 3.5.1-1)}
\]

\[
t_c = \text{concrete compressive strength (ksi)} \\
\gamma_e = 0.75 \text{ for Class 2 Exposure. C5.6.7 defines Class 2 Exposure as decks and any substructure units exposed to water.}
\]

**Check Minimum Reinforcement** \quad \text{(5.6.3.3)}

The minimum reinforcement requirements state:

\[
M_r = \phi M_n > \min(M_{cr}, 1.33M_{\text{STRENGTH}})
\]

Where:

\[
M_{cr} = \gamma_3 \gamma_1 S f_r \text{ (k-in.)} \quad \text{(Eq. 5.6.3.3-1)}
\]

\[
S = \frac{1}{6}bh^2 \text{ (in.}^3\text{)}
\]

\[
f_r = 0.24\sqrt{f'_c} \text{ (ksi)} \quad \text{(5.4.2.6)}
\]

\[
\gamma_3 = 0.75 \text{ for A706, Grade 60 reinforcement} \\
\gamma_1 = 1.6 \text{ for non-segmentally constructed bridges}
\]
Design Guides 3.2.1 - LRFD Deck Design

LRFD Deck Slab Design Example: 7 ft. Beam Spacing, Positive Moment Reinforcement

Design Stresses

\[ f_y = 60 \text{ ksi} \]
\[ f'_c = 4 \text{ ksi} \]

Design Thickness

Standard eight inch slab thickness.

Determine Maximum Factored Loading

Unfactored Loads and Moments

\[ w_{DC1} = \left( \frac{0.150 \text{ k}}{\text{ft}^3} \right) (0.667 \text{ ft})(1 \text{ ft}) = 0.100 \frac{\text{k}}{\text{ft}} \]
\[ w_{DW} = \left( \frac{0.050 \text{ k}}{\text{ft}^2} \right)(1 \text{ ft}) = 0.050 \frac{\text{k}}{\text{ft}} \]
\[ M_{DC1} = \frac{1}{10} \left( \frac{0.100 \text{k}}{\text{ft}} \right)(7 \text{ ft})^2 = 0.490 \text{ k-ft.} \]
\[ M_{DW} = \frac{1}{10} \left( \frac{0.050 \text{k}}{\text{ft}} \right)(7 \text{ ft})^2 = 0.245 \text{ k-ft.} \]
\[ M_{LL+IM} = 5.21 \text{ k-ft.} \]

Factored Moments (Appendix A4)

\[ M_{STRENGTH_I} = \eta [1.25M_{DC1} + 1.5M_{DW} + 1.75M_{LL+IM}] \]
\[ = 1.00[1.25(0.490 \text{ k-ft.}) + 1.5(0.245 \text{ k-ft.}) + 1.75(5.21 \text{ k-ft.})] \]
\[ = 10.10 \text{ k-ft.} \left( \frac{12 \text{ in.}}{\text{ft.}} \right) \]
\[ = 121.17 \text{ k-in.} \]
$$M_{\text{SERVICE}} = \eta \left[ 1.0M_{\text{DC1}} + 1.0M_{\text{DW}} + 1.0M_{\text{LL+HM}} \right]$$

$$= 1.00 \left[ 1.0(0.490 \text{ k-ft.}) + 1.0(0.245 \text{ k-ft.}) + 1.0(5.21 \text{ k-ft.}) \right]$$

$$= 5.95 \text{ k-ft.} \left( \frac{12 \text{ in.}}{12 \text{ ft.}} \right)$$

$$= 71.34 \text{ k-in.}$$

**Check Flexural Resistance** \((5.6.3.2)\)

$$M_r = \phi M_n = \phi \left[ A_s f_s \left( d_s - \frac{a}{2} \right) \right] \geq M_{\text{STRENGTH}} \quad \text{ (Eqs. 5.6.3.2.1-1 & 5.6.3.2.2-1)}$$

Using #5 bars, solve for \(A_s\):

- \(b = 12 \text{ in.}\)
- \(d_s = 8 \text{ in.} - 1 \text{ in. clear} - 0.5(0.625 \text{ in. bar diameter}) = 6.69 \text{ in.}\)
- \(f_s = \text{Assume 60 ksi, if } c / d_s < 0.6 \text{ then assumption is valid \ (5.6.2.1)}\)
- \(f'_c = 4 \text{ ksi}\)
- \(\phi = \text{Assumed to be 0.9, then checked below}\)
- \(\beta_1 = 0.85 \quad \text{(5.6.2.2)}\)
- \(c = \frac{A_s (60 \text{ ksi})}{0.85 (0.85)(4 \text{ ksi})(12 \text{ in.})} = 1.73A_s \text{ in.} \quad \text{(Eq. 5.6.3.1.1-4)}\)
- \(a = \beta_1 c = 0.85(1.73A_s) = 1.47A_s \text{ in.}\)
- \(M_r = M_{\text{STRENGTH}} = 121.17 \text{ k-in.}\)
- \(121.17 \text{ k-in.} = (0.9) \left[ A_s (60 \text{ ksi}) \left( 6.69 \text{ in.} - \frac{1.47A_s \text{ in.}}{2} \right) \right] \)

Solving for \(A_s\) gives \(A_s = 0.35 \text{ in.}^2\). Try #5 bars @ 10 in. center-to-center spacing, \(A_s = 0.37 \text{ in.}^2\)

Check \(\frac{c}{d_s} < 0.6\) to validate \(f_s = f_y\) assumption:

- \(c = 1.73 A_s = 1.73(0.37 \text{ in.}^2) = 0.64 \text{ in.}\)
\[ ds = 6.69 \text{ in.} \]

\[ \frac{c}{ds} = \frac{0.64 \text{ in.}}{6.69 \text{ in.}} = 0.1 < 0.6 \quad \therefore \text{Assumption of } f_s = f_y = 60 \text{ ksi is valid.} \]

Verify \( \phi = 0.9 \) assumption:

\[ \varepsilon_t = \frac{0.003(d_t - c)}{c} \quad (C5.6.2.1-1) \]

Where:

\[ c = 0.64 \text{ in.} \]
\[ d_t = d_s = 6.69 \text{ in.} \]

\[ \varepsilon_t = \frac{0.003(6.69 \text{ in.} - 0.64 \text{ in.})}{0.64 \text{ in.}} = 0.028 \]

\[ 0.028 > 0.005, \quad \therefore \text{Assumption of } \phi = 0.9 \text{ is valid.} \]

Check Control of Cracking \quad (5.6.7)

\[ s \leq \frac{700 \gamma_o}{\beta_s f_{ss}} - 2d_c \quad (\text{Eq. 5.6.7-1}) \]

Where:

\[ d_c = 1 \text{ in. clear} + 0.5(0.625 \text{ in. bar diameter}) = 1.313 \text{ in.} \]
\[ h = 8 \text{ in.} \]

\[ \beta_s = 1 + \frac{1.313 \text{ in.}}{0.7(8 \text{ in.} - 1.313 \text{ in.})} = 1.28 \]

\[ \rho = \frac{0.37 \text{ in.}^2}{(12 \text{ in.})(6.69 \text{ in.})} = 0.00461 \]

\[ n = \frac{E_s}{E_c} \]

\[ E_s = 29000 \text{ ksi} \quad (6.4.1) \]

\[ E_c = 120000K_iw_c^{2f'_c 0.33} \quad (\text{Eq. 5.4.2.4-1}) \]
\[ = 120000(1.0)(0.145 \text{ kcf})^{2}(4 \text{ ksi})^{0.33} \]
\[
\begin{align*}
\text{n} &= \frac{29000 \text{ ksi}}{3987 \text{ ksi}} = 7.27 \\
k &= \sqrt{[(0.00461)(7.27)]^2 + 2(0.00461)(7.27) - (0.00461)(7.27)} = 0.228 \\
j &= 1 - \frac{0.228}{3} = 0.924 \\
f_{ss} &= \frac{(71.34 \text{ k-in.})}{(0.37 \text{ in.}^2)(0.924)(6.69 \text{ in.})} = 31.19 \text{ ksi} < 0.6f_y = 36 \text{ ksi} \quad \text{O.K.}
\end{align*}
\]

\[
\begin{align*}
\gamma_e &= 0.75 \\
\frac{700\gamma_e}{\beta_s f_{s}} - 2d_c &= \frac{700(0.75)}{(1.28)(31.19)} - 2(1.313) = 10.52 \text{ in.}
\end{align*}
\]

\[s = 10 \text{ in.} < 10.52 \text{ in.} \quad \text{O.K.}
\]

\(\therefore \text{#5 bars @ 10 in. center-to-center spacing is adequate to control cracking.}\)

**Check Minimum Reinforcement**

\[
M_r = \phi M_n > \min(M_{cr}, 1.33M_{\text{STRENGTH I}})
\]

Where:

\[
M_{cr} = \gamma_3 \gamma_1 S f_r \quad \text{(k-in.)} \quad \text{(Eq. 5.6.3.3-1)}
\]

\[
\gamma_3 = 0.75 \text{ for A706, Grade 60 reinforcement}
\]

\[
\gamma_1 = 1.6 \text{ for non-segmentally constructed bridges}
\]

\[
S = \frac{1}{6}(12 \text{ in.})(8 \text{ in.})^2 = 128 \text{ in.}^3
\]

\[
f_r = 0.24\sqrt{4 \text{ ksi}} = 0.48 \text{ ksi} \quad \text{(5.4.2.6)}
\]

\[
M_{cr} = 0.75(1.6)(128 \text{ in.}^3)(0.48 \text{ ksi}) = 73.7 \text{ k-in.}
\]

\[
1.33M_{\text{STRENGTH I}} = 1.33(121.17 \text{ k-in.}) = 161.1 \text{ k-in.}
\]
LRFD Deck Slab Design Example (continued): 7 ft. Beam Spacing, Negative Moment Reinforcement

Determine Maximum Factored Loading

Unfactored Loads and Moments:

\[
\begin{align*}
\mathbf{w}_{DC1} &= \frac{0.150 \text{k}}{\text{ft}^3} \times (0.667 \text{ ft})(1 \text{ ft}) = 0.100 \frac{\text{k}}{\text{ft}} \\
\mathbf{w}_{DW} &= \frac{0.050 \text{k}}{\text{ft}^2} \times (1 \text{ ft}) = 0.050 \frac{\text{k}}{\text{ft}}
\end{align*}
\]

Assuming steel girders with twelve inch top flange widths, the span length shall be reduced by six inches as shown in Bridge Manual Figure 3.2.1-2 for the negative moment region.

\[
\begin{align*}
\mathbf{M}_{DC1} &= \frac{1}{10} \left(0.100 \frac{\text{k}}{\text{ft}}\right)(6.5 \text{ ft})^2 = 0.423 \text{k-ft.} \\
\mathbf{M}_{DW} &= \frac{1}{10} \left(0.050 \frac{\text{k}}{\text{ft}}\right)(6.5 \text{ ft})^2 = 0.211 \text{k-ft.} \\
\mathbf{M}_{LL+IM} &= 5.17 \text{k-ft.}
\end{align*}
\]

Note: The 5.17 k-ft. moment corresponds to the value found in Appendix A4 for a section taken three inches from the centerline of beam, for a seven foot beam spacing.

Factored Moments (Table 3.4.1-1)
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3.2.1 - LRFD Deck Design

\[ M_{\text{STRENGTH I}} = \eta \left[ 1.25M_{DC1} + 1.5M_{DW} + 1.75M_{LL+IM} \right] \]

= 1.00 \left[ 1.25(0.423 \text{ k-ft.}) + 1.5(0.211 \text{ k-ft.}) + 1.75(5.17 \text{ k-ft.}) \right]

= 9.89 \text{ k-ft.} \left( \frac{12 \text{ in.}}{\text{ft.}} \right)

= 118.71 \text{ k-in.}

\[ M_{\text{SERVICE I}} = \eta \left[ 1.0M_{DC1} + 1.0M_{DW} + 1.0M_{LL+IM} \right] \]

= 1.00 \left[ 1.0(0.423 \text{ k-ft.}) + 1.0(0.211 \text{ k-ft.}) + 1.0(5.17 \text{ k-ft.}) \right]

= 5.80 \text{ k-ft.} \left( \frac{12 \text{ in.}}{\text{ft.}} \right)

= 69.65 \text{ k-in.}

**Check Flexural Resistance**

(Eqs. 5.6.3.2.1-1 & 5.6.3.2.2-1)

\[ M_r = \phi M_n = \phi \left[ A_s f_s \left( d_s - \frac{a}{2} \right) \right] \geq M_{\text{STRENGTH I}} \]

Using #5 bars, solve for \( A_s \):

\[ b = 12 \text{ in.} \]

\[ d_s = 8 \text{ in.} - 2.5 \text{ in. clear} - 0.5(0.625 \text{ in. bar diameter}) = 5.19 \text{ in.} \]

\[ f_s = \text{Assume 60 ksi, if } c/d_s < 0.6 \text{ then assumption is valid} \quad (5.6.2.1) \]

\[ f'_c = 4 \text{ ksi} \]

\[ \phi = \text{Assumed to be 0.9, then checked below} \]

\[ \beta_1 = 0.85 \quad (5.6.2.2) \]

\[ c = \frac{A_s (60 \text{ ksi})}{0.85(0.85)(4 \text{ ksi})(12 \text{ in.})} = 1.73A_s \text{ in.} \]

(Eq. 5.6.3.1.1-4)

\[ a = \beta_1 c = 0.85(1.73A_s) = 1.47A_s \text{ in.} \]

\[ M_r = M_{\text{STRENGTH I}} = 118.71 \text{ k-in.} \]

\[ 118.71 \text{ k-in.} = (0.9) \left[ A_s (60 \text{ ksi}) \left( 5.19 \text{ in.} - \frac{1.47A_s \text{ in.}}{2} \right) \right] \]

Solving for \( A_s \) gives \( A_s = 0.45 \text{ in.}^2 \) Try #5 bars @ 8 in. center-to-center spacing, \( A_s = 0.46 \text{ in.}^2 \).
Check $\frac{c}{d_s} < 0.6$ to validate $f_s = f_y$ assumption:

\[ c = 1.73 A_s = 1.73(0.46 \text{ in.}^2) = 0.80 \text{ in.} \]
\[ d_s = 5.19 \text{ in.} \]

\[ \frac{c}{d_s} = \frac{0.80 \text{ in.}}{5.19 \text{ in.}} = 0.15 < 0.6 \quad \therefore \text{Assumption of } f_s = f_y = 60 \text{ ksi is valid.} \]

Verify $\phi = 0.9$ assumption:

\[ \varepsilon_t = \frac{0.003(d_t - c)}{c} \]  \hspace{1cm} (C5.6.2.1-1)

Where:

\[ c = 0.80 \text{ in.} \]
\[ d_t = d_s = 5.19 \text{ in.} \]

\[ \varepsilon_t = \frac{0.003(5.19 \text{ in.} - 0.80 \text{ in.})}{0.80 \text{ in.}} = 0.016 \]

\[ 0.016 > 0.005, \quad \therefore \text{Assumption of } \phi = 0.9 \text{ is valid.} \]

**Check Control of Cracking**  \hspace{1cm} (5.6.7)

\[ s \leq \frac{700 \gamma_c}{\beta_s f_{ss}} - 2d_c \]  \hspace{1cm} (Eq. 5.6.7-1)

Where:

\[ d_c = 2.5 \text{ in. clear} + 0.5(0.625 \text{ in. bar diameter}) = 2.813 \text{ in.} \]
\[ h = 8 \text{ in.} \]
\[ \beta_s = 1 + \frac{2.813 \text{ in.}}{0.7(8 \text{ in.} - 2.813 \text{ in.})} = 1.775 \]
\[ \rho = \frac{0.46 \text{ in.}^2}{(12 \text{ in.})(5.19 \text{ in.})} = 0.00739 \]
\[ n = \frac{E_s}{E_c} \]

\[ E_s = 29000 \text{ ksi} \quad (6.4.1) \]

\[ E_c = 120000K_1w_c2\bar{f}_c^{0.33} \quad (\text{Eq. 5.4.2.4-1}) \]

\[ = 120000(1.0)(0.145 \text{ ksi})^2(4 \text{ ksi})^{0.33} \]

\[ = 3987 \text{ ksi} \]

\[ n = \frac{29000 \text{ ksi}}{3987 \text{ ksi}} \]

\[ = 7.27 \]

\[ k = \sqrt{(0.00739)(7.27)^2 + 2(0.00739)(7.27) - (0.00739)(7.27)} \]

\[ = 0.278 \]

\[ j = 1 - \frac{0.278}{3} = 0.907 \]

\[ f_{ss} = \frac{69.65 \text{ k in.}}{(0.46 \text{ in.}^2)(0.907)(5.19 \text{ in.})} = 32.17 \text{ ksi} < 0.6f_y = 36 \text{ ksi} \quad \text{O.K.} \]

\[ \gamma_e = 0.75 \]

\[ \frac{700\gamma_e}{\beta_s f_s} - 2d_c = \frac{700(0.75)}{(1.775)(30.22)} - 2(2.813) = 4.16 \text{ in.} \]

\[ s = 8 \text{ in.} > 4.16 \text{ in.} \quad \text{N.G.} \]

Try 6 in. spacing., \( A_s = 0.61 \text{ in.}^2 \)

\[ s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad (\text{Eq. 5.6.7-1}) \]

Where:

\[ d_c = 2.5 \text{ in. clear} + 0.5(0.625 \text{ in. bar diameter}) = 2.813 \text{ in.} \]

\[ h = 8 \text{ in.} \]

\[ \beta_s = 1 + \frac{2.813 \text{ in.}}{0.7(8 \text{ in.} - 2.813 \text{ in.})} = 1.775 \]

\[ \rho = \frac{0.61 \text{ in.}^2}{(12 \text{ in.})(5.19 \text{ in.})} = 0.0098 \]
Design Guides

3.2.1 - LRFD Deck Design

\[ n = \frac{E_s}{E_c} \]
\[ E_s = 29000 \text{ ksi} \quad \text{(6.4.1)} \]
\[ E_c = 120000K_i w_c^2 f_i^{0.33} \quad \text{(Eq. 5.4.2.4-1)} \]
\[ = 120000(1.0)(0.145 \text{ kcf})^2(4 \text{ ksi})^{0.33} \]
\[ = 3987 \text{ ksi} \]
\[ n = \frac{29000 \text{ ksi}}{3987 \text{ ksi}} \]
\[ = 7.27 \]
\[ k = \sqrt{[0.0098)(7.27)]^2 + 2(0.0098)(7.27) - (0.0098)(7.27) = 0.313} \]
\[ j = 1 - \frac{0.313}{3} = 0.896 \]
\[ f_{ss} = \frac{(69.65 \text{ k-in.})}{(0.61 \text{ in.}^2)(0.896)(5.19 \text{ in.})} = 24.55 \text{ ksi} < 0.6f_y = 36 \text{ ksi} \quad \text{O.K.} \]
\[ \gamma_e = 0.75 \]
\[ \frac{700\gamma_e}{\beta_s f_s} - 2d_c = \frac{700(0.75)}{(1.775)(24.55)} - 2(2.813) = 6.42 \text{ in.} \]
\[ s = 6 \text{ in.} < 6.42 \text{ in.} \quad \text{O.K.} \]

:\ #5 bars @ 6 in. center-to-center spacing is adequate to control cracking.

Check Minimum Reinforcement \quad (5.6.3.3)

\[ M_r = \phi M_n > \min(M_{cr}, 1.33M_{\text{STRENGTH}_i}) \]

Where:
\[ M_{cr} = \gamma_3 \gamma_i S f_r \text{ (k-in.)} \quad \text{(Eq. 5.6.3.3-1)} \]
\[ \gamma_3 = 0.75 \text{ for A706, Grade 60 reinforcement} \]
\[ \gamma_i = 1.6 \text{ for non-segmentally constructed bridges} \]
\[ S = \frac{1}{6} (12 \text{ in.})(8 \text{ in.})^2 = 128 \text{ in.}^3 \]
\[ f_r = 0.24\sqrt{4 \text{ ksi}} = 0.48 \text{ ksi} \quad \text{(5.4.2.6)} \]
\[ M_{cr} = 0.75(1.6)(128 \text{ in.}^3)(0.48 \text{ ksi}) = 73.7 \text{ k-in.} \]

\[ 1.33M_{\text{STRENGTH I}} = 1.33(118.71 \text{ k-in.}) = 157.9 \text{ k-in.} \]

\[
\min(M_{cr}, M_{\text{STRENGTH I}}) = 73.7 \text{ k-in.}
\]

\[
\phi M_n = 0.9\left(0.61 \text{ in.}^2\right)(60 \text{ ksi}) \left[ \frac{5.19 \text{ in.} - \left(\frac{0.61 \text{ in.}^2}{2(0.85)(12 \text{ in.})}(60 \text{ ksi})\right)}{2(0.85)(12 \text{ in.})(4 \text{ ksi})} \right] = 156.18 \text{ k-in.}
\]

156.18 k-in. > 73.7 k-in.  

O.K.

Summary:

Use #5 bars @ 10 in. center-to-center spacing for positive moment reinforcement.  
Use #5 bars @ 6 in. center-to-center spacing for negative moment reinforcement.