Illinois Department of Transportation
Office of Highways Project Implementation / Bureau of Bridges & Structures
2300 South Dirksen Parkway / Springfield, Illinois / 62764

All IDOT Design Guides have been updated to reflect the release of the 2017 AASHTO LRFD Bridge Design Specification, 8^{th} 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, λ. 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 α_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 ksi
f_y = 60 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:

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

Where γ_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:

$M_{SERVICE\ I} = 1.0(DC + DW + LL + IM)$ (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 η_i, defined as:

$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$ (1.3.2.1-2)

Where:

η_D = ductility factor, taken as 1.00 for conventional designs

η_R = redundancy factor, taken as 1.00 for conventional levels of redundancy

η_I = 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 (k-in.), is taken as:

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

Where:

ϕ = Assumed to be 0.9, then checked using the procedure found in Article 5.5.4.2. In this procedure, the reinforcement strain, ε_t, is calculated, and ϕ is dependent upon this strain. ε_t 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 $0.002 < \varepsilon_t < 0.005$, $\phi = 0.75 + \dfrac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})}$
- If $\varepsilon_t > 0.005$, $\phi = 0.9$

Where ε_{cl} is taken as 0.002 and ε_{tl} is taken as 0.005, as stated in Article 5.6.2.1.

a = depth of equivalent stress block (in.), taken as $a = \beta_1 c$

$$c = \frac{A_s f_s}{\alpha_1 \beta_1 f'_c\, b} \text{ (in.)} \quad \text{(Eq. 5.6.3.1.1-4)}$$

A_s = area of tension reinforcement in strip (in.^{2})

b = width of design strip (in.)

d_s = distance from extreme compression fiber to centroid of tensile reinforcement (in.)

f_s = stress in the mild steel tension reinforcement as specified at nominal flexural resistance (ksi). As specified in Article 5.6.2.1, if $c / d_s < 0.003 / (0.003 + \varepsilon_{cl})$, then f_y may used in lieu of exact computation of f_s. For 60 ksi reinforcement, ε_{cl} is taken as 0.002, making the ratio $0.003 / (0.003 + \varepsilon_{cl})$ equal to 0.6. Typically in design, f_s is assumed to be equal to f_y, then the assumption is checked.

f'_c = specified compressive strength of concrete (ksi)

α_1 = 0.85 for concrete with strength less than 10 ksi (5.6.2.2)

β_1 = 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:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(Eq. 5.6.7-1)}$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \qquad \text{(Eq. 5.6.7-2)}$$

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

h = slab depth (in.)

f_{ss} = stress in mild steel tension reinforcement at service load condition, not to exceed $0.6f_y$

$= \dfrac{M_{SERVICE\ I}}{A_s j d_s}$ (ksi)

$j = 1 - \dfrac{k}{3}$

$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$

$\rho = \dfrac{A_s}{bd_s}$

$n = \dfrac{E_s}{E_c}$

E_s = 29000 ksi (6.4.1)

E_c = $120000K_1 w_c^2 f'_c{}^{0.33}$ (Eq. 5.4.2.4-1)

K_1 = 1.0 for normal-weight concrete

w_c = 0.145 kcf (Table 3.5.1-1)

f'_c = concrete compressive strength (ksi)

γ_e = 0.75 for Class 2 Exposure. C5.6.7 defines Class 2 Exposure as decks and any substructure units exposed to water.

Check Minimum Reinforcement *(5.6.3.3)*

The minimum reinforcement requirements state:

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

Where:

$M_{cr} = \gamma_3 \gamma_1 S f_r$ (k-in.) (Eq. 5.6.3.3-1)

$S = \dfrac{1}{6} bh^2$ (in.^{3})

$f_r = 0.24\sqrt{f'_c}$ (ksi) (5.4.2.6)

γ_3 = 0.75 for A706, Grade 60 reinforcement

γ_1 = 1.6 for non-segmentally constructed bridges

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

Design Stresses

f_y = 60 ksi

f'_c = 4 ksi

Design Thickness

Standard eight inch slab thickness.

Determine Maximum Factored Loading

Unfactored Loads and Moments

$$w_{DC1} = \left(\frac{0.150\text{ k}}{\text{ft.}^3}\right)(0.667\text{ ft.})(1\text{ ft.}) = 0.100\frac{\text{k}}{\text{ft.}}$$

$$w_{DW} = \left(\frac{0.050\text{ k}}{\text{ft.}^2}\right)(1\text{ ft.}) = 0.050\frac{\text{k}}{\text{ft.}}$$

$$M_{DC1} = \frac{1}{10}\left(0.100\frac{\text{k}}{\text{ft.}}\right)(7\text{ ft.})^2 = 0.490\text{ k-ft.}$$

$$M_{DW} = \frac{1}{10}\left(0.050\frac{\text{k}}{\text{ft.}}\right)(7\text{ ft.})^2 = 0.245\text{ k-ft.}$$

M_{LL+IM} = 5.21 k-ft. (Appendix A4)

Factored Moments (Table 3.4.1-1)

$$M_{STRENGTH\ I} = \eta_i[1.25M_{DC1} + 1.5M_{DW} + 1.75M_{LL+IM}]$$

$$= 1.00[1.25(0.490\text{ k-ft.}) + 1.5(0.245\text{ k-ft.}) + 1.75(5.21\text{ k-ft.})]$$

$$= 10.10\text{ k-ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)$$

$$= 121.17\text{ k-in.}$$

$M_{SERVICE\ I} = \eta_i[1.0M_{DC1} + 1.0M_{DW} + 1.0M_{LL+IM}]$

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

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

$= 71.34$ k-in.

Check Flexural Resistance *(5.6.3.2)*

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

Using #5 bars, solve for A_s:

b = 12 in.

d_s = 8 in. – 1 in. clear – 0.5(0.625 in. bar diameter) = 6.69 in.

f_s = Assume 60 ksi, if $c / d_s < 0.6$ then assumption is valid (5.6.2.1)

f'_c = 4 ksi

ϕ = Assumed to be 0.9, then checked below

β_1 = 0.85 (5.6.2.2)

$c = \frac{A_s(60 \text{ ksi})}{0.85(0.85)(4 \text{ ksi})(12 \text{ in.})} = 1.73A_s$ in. (Eq. 5.6.3.1.1-4)

$a = \beta_1 c = 0.85(1.73A_s) = 1.47A_s$ in.

$M_r = M^{+}_{STRENGTH\ I} = 121.17$ 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 $A_s = 0.35$ in.^{2} Try #5 bars @ 10 in. center-to-center spacing, A_s = 0.37 in.^{2}

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

$c = 1.73\ A_s = 1.73(0.37 \text{ in.}^2) = 0.64$ in.

d_s = 6.69 in.

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

Verify ϕ = 0.9 assumption:

$$\varepsilon_t = \frac{0.003(d_t - c)}{c} \qquad \text{(C5.6.2.1-1)}$$

Where:

c = 0.64 in.

d_t = d_s = 6.69 in.

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

0.028 > 0.005, $\therefore$ Assumption of ϕ = 0.9 is valid.

Check Control of Cracking *(5.6.7)*

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(Eq. 5.6.7-1)}$$

Where:

d_c = 1 in. clear + 0.5(0.625 in. bar diameter) = 1.313 in.

h = 8 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 ksi (6.4.1)

E_c = $120000K_1w_c^2f'^{0.33}_c$ (Eq. 5.4.2.4-1)

= $120000(1.0)(0.145\text{ kcf})^2(4\text{ ksi})^{0.33}$

$= 3987\text{ ksi}$

$$n = \frac{29000\text{ ksi}}{3987\text{ ksi}}$$

$= 7.27$

$$k = \sqrt{[(0.00461)(7.27)]^2 + 2(0.00461)(7.27)} - (0.00461)(7.27) = 0.228$$

$$j = 1 - \frac{0.228}{3} = 0.924$$

$$f_{ss} = \frac{(71.34\text{ k}-\text{in.})}{(0.37\text{ in.}^2)(0.924)(6.69\text{ in.})} = 31.19\text{ ksi} < 0.6f_y = 36\text{ ksi} \quad \text{O.K.}$$

$\gamma_e = 0.75$

$$\frac{700\gamma_e}{\beta_s f_s} - 2d_c = \frac{700(0.75)}{(1.28)(31.19)} - 2(1.313) = 10.52\text{ in.}$$

$s = 10\text{ in.} < 10.52\text{ in.}$ O.K.

$\therefore$ #5 bars @ 10 in. center-to-center spacing is adequate to control cracking.

Check Minimum Reinforcement *(5.6.3.3)*

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

Where:

$M_{cr} = \gamma_3\gamma_1 S f_r$ (k-in.) (Eq. 5.6.3.3-1)

γ_3 = 0.75 for A706, Grade 60 reinforcement

γ_1 = 1.6 for non-segmentally constructed bridges

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

$f_r = 0.24\sqrt{4\text{ ksi}} = 0.48\text{ ksi}$ (5.4.2.6)

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

$1.33M_{STRENGTH\ I} = 1.33(121.17\text{ k-in.}) = 161.1\text{ k-in.}$

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

$$\phi M_n = 0.9\left(0.37\text{ in.}^2\right)\left(60\text{ ksi}\right)\left[6.69\text{ in.} - \frac{\left(0.37\text{ in.}^2\right)\left(60\text{ ksi}\right)}{2\left(0.85\right)\left(12\text{ in.}\right)\left(4\text{ ksi}\right)}\right] = 126.9\text{ k-in.}$$

126.9 k-in. > 73.7 k-in. O.K.

LRFD Deck Slab Design Example (continued): 7 ft. Beam Spacing, Negative Moment Reinforcement

Determine Maximum Factored Loading

<u>Unfactored Loads and Moments:</u>

$$w_{DC1} = \left(\frac{0.150\text{ k}}{\text{ft.}^3}\right)\left(0.667\text{ ft.}\right)\left(1\text{ ft.}\right) = 0.100\frac{\text{k}}{\text{ft.}}$$

$$w_{DW} = \left(\frac{0.050\text{ k}}{\text{ft.}^2}\right)\left(1\text{ ft.}\right) = 0.050\frac{\text{k}}{\text{ft.}}$$

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.

$$M_{DC1} = \frac{1}{10}\left(0.100\frac{\text{k}}{\text{ft.}}\right)\left(6.5\text{ ft.}\right)^2 = 0.423\text{ k-ft.}$$

$$M_{DW} = \frac{1}{10}\left(0.050\frac{\text{k}}{\text{ft.}}\right)\left(6.5\text{ ft.}\right)^2 = 0.211\text{ k-ft.}$$

M_{LL+IM} = 5.17 k-ft.

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.

<u>Factored Moments</u> (Table 3.4.1-1)

$$M_{STRENGTH\ I} = \eta_i[1.25M_{DC1} + 1.5M_{DW} + 1.75M_{LL+IM}]$$

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

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

$$= 118.71\text{ k-in.}$$

$$M_{SERVICE\ I} = \eta_i[1.0M_{DC1} + 1.0M_{DW} + 1.0M_{LL+IM}]$$

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

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

$$= 69.65\text{ k-in.}$$

Check Flexural Resistance *(5.6.3.2)*

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

Using #5 bars, solve for A_s:

b = 12 in.

d_s = 8 in. – 2.5 in. clear – 0.5(0.625 in. bar diameter) = 5.19 in.

f_s = Assume 60 ksi, if $c / d_s < 0.6$ then assumption is valid (5.6.2.1)

f'_c = 4 ksi

ϕ = Assumed to be 0.9, then checked below

β_1 = 0.85 (5.6.2.2)

$c = \dfrac{A_s(60\text{ ksi})}{0.85(0.85)(4\text{ ksi})(12\text{ in.})} = 1.73A_s$ in. (Eq. 5.6.3.1.1-4)

$a = \beta_1 c = 0.85(1.73A_s) = 1.47A_s$ in.

$M_r = M^-_{STRENGTH\ I} = 118.71$ 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 $A_s = 0.45$ in.^{2} Try #5 bars @ 8 in. center-to-center spacing, A_s = 0.46 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:

$$\varepsilon_t = \frac{0.003(d_t - c)}{c} \qquad \text{(C5.6.2.1-1)}$$

Where:

$c = 0.80\ \text{in.}$

$d_t = d_s = 5.19\ \text{in.}$

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

$0.016 > 0.005$, $\therefore$ Assumption of $\phi = 0.9$ is valid.

Check Control of Cracking *(5.6.7)*

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(Eq. 5.6.7-1)}$$

Where:

d_c = 2.5 in. clear + 0.5(0.625 in. bar diameter) = 2.813 in.

h = 8 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 ksi (6.4.1)

E_c = $120000K_1w_c^2f'_c{}^{0.33}$ (Eq. 5.4.2.4-1)

= $120000(1.0)(0.145\text{ kcf})^2(4\text{ ksi})^{0.33}$

= 3987 ksi

$$n = \frac{29000\text{ ksi}}{3987\text{ ksi}}$$

= 7.27

$$k = \sqrt{[(0.00739)(7.27)]^2 + 2(0.00739)(7.27)} - (0.00739)(7.27) = 0.278$$

$$j = 1 - \frac{0.278}{3} = 0.907$$

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

γ_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 in. > 4.16 in. N.G.

Try 6 in. spacing., A_s = 0.61 in.^{2}

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$ (Eq. 5.6.7-1)

Where:

d_c = 2.5 in. clear + 0.5(0.625 in. bar diameter) = 2.813 in.

h = 8 in.

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

$$\rho = \frac{0.61\text{ in.}^2}{(12\text{ in.})(5.19\text{ in.})} = 0.0098$$

$$n = \frac{E_s}{E_c}$$

E_s = 29000 ksi (6.4.1)

E_c = $120000K_1w_c^2f'_c{}^{0.33}$ (Eq. 5.4.2.4-1)

= $120000(1.0)(0.145\text{ kcf})^2(4\text{ ksi})^{0.33}$

= 3987 ksi

$$n = \frac{29000\text{ ksi}}{3987\text{ ksi}}$$

= 7.27

$$k = \sqrt{[(0.0098)(7.27)]^2 + 2(0.0098)(7.27)} - (0.0098)(7.27) = 0.313$$

$$j = 1 - \frac{0.313}{3} = 0.896$$

$$f_{ss} = \frac{(69.65\text{ k} - \text{in.})}{(0.61\text{ in.}^2)(0.896)(5.19\text{ in.})} = 24.55\text{ ksi} < 0.6f_y = 36\text{ ksi} \quad \text{O.K.}$$

γ_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 in. < 6.42 in. O.K.

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

Check Minimum Reinforcement *(5.6.3.3)*

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

Where:

$M_{cr} = \gamma_3\gamma_1 S f_r$ (k-in.) (Eq. 5.6.3.3-1)

γ_3 = 0.75 for A706, Grade 60 reinforcement

γ_1 = 1.6 for non-segmentally constructed bridges

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

$f_r = 0.24\sqrt{4\text{ ksi}} = 0.48\text{ ksi}$ (5.4.2.6)

$M_{cr} =$ 0.75(1.6)(128 in.^{3})(0.48 ksi) = 73.7 k-in.

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

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

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

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.