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, 8th Edition. The following is a summary of the major changes that have been incorporated into the Slab Bridge 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.
- Parapets, curbs, and railings are now to be included in the calculation of the DC load case.
- The load factors for Fatigue I and Fatigue II have been increased to 1.75 and 0.8, respectively.
- 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.
- The cover for bottom longitudinal bars has been increased from 1” to 1.5” in accordance with ABD 15.4.
- Limits of Reinforcement have been changed to Minimum Reinforcement and all of the sections relating to Maximum Reinforcement have been removed.
- The additional Department requirements for slab bridges from ABD 15.8 have been added.
- Additional instruction has been added for the design of edge beams.

- The value of β in the concrete resistance equation is now an exact value as opposed to being conservatively taken as 2. The procedure for this process has been added.

- The shear steel resistance is no longer simplified by section 5.8.3.4.1. The procedure for how it is to be calculated has been included.

- 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.)

3.2.11 LRFD Slab Bridge Design

Slab bridges are defined as structures where the deck slab also serves as the main load-carrying component. This design guide provides a basic procedural outline for the design of slab bridges using the LRFD Code and also includes a worked example. Unless otherwise specified, all code references refer to the AASHTO LRFD Bridge Design Specifications, 8^{th} Ed.

Main reinforcement in slab bridges is designed for Flexural Resistance (5.6.3.2), Fatigue (5.5.3), Control of Cracking (5.6.7), and Minimum Reinforcement (5.6.3.3). All reinforcement shall be fully developed at the point of necessity. The minimum slab depth guidelines specified in Table 2.5.2.6.3-1 need not be followed if the reinforcement meets these requirements.

For design, the Approximate Elastic or “Strip” Method for slab bridges found in Article 4.6.2.3 shall be used.

According to Article 9.7.1.4, edges of slabs shall either be strengthened or be supported by an edge beam which is integral with the slab. As depicted in Figure 3.2.11-1 of the Bridge Manual, the reinforcement which extends from the concrete barrier into the slab qualifies as shear reinforcement (strengthening) for the outside edges of slabs. When a concrete barrier is used on a slab bridge, its structural adequacy as an edge beam should typically only need to be verified. The barrier itself should not be considered structural- only the vertical reinforcement extending into the slab. Edge beam design is required for bridges with open joints and possibly at stage construction lines. If the out-to-out width of a slab bridge exceeds 45 ft., an open longitudinal joint is required.

LRFD Slab Bridge Design Procedure, Equations, and Outline

Determine Live Load Distribution Factor *(4.6.2.3)*

Live Load distribution factors are calculated by first finding the equivalent width per lane that will be affected. This equivalent strip width, in inches, is found using the following equations:

For single-lane loading or two lines of wheels (e.g. used for staged construction design considerations where a single lane of traffic is employed), the strip width E is taken as:

$$E = 10.0 + 5.0\sqrt{L_1 W_1} \qquad \text{(Eq. 4.6.2.3-1)}$$

For multiple-lane loading, the strip width E is taken as:

$$E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \frac{12.0W}{N_L} \qquad \text{(Eq. 4.6.2.3-2)}$$

When calculating E:

L_1 = modified span length, taken as the lesser of (a) the actual span length (ft.) or (b) 60 ft.

N_L = number of design lanes according to Article 3.6.1.1.1

W = actual edge-to-edge width of bridge (ft.)

W_1 = modified edge-to-edge width of bridge, taken as the lesser of (a) the actual edge to edge width W (ft.), or (b) 60 ft. for multiple-lane loading, 30 ft. for single-lane loading

According to Article 3.6.1.1.2, multiple presence factors shall not be employed when designing bridges utilizing Equations 4.6.2.3-1 and 4.6.2.3-2 as they are already embedded in the formulae.

The fatigue truck loading specified in Article 3.6.1.4 is distributed using the single-lane loaded strip width given in Equation 4.6.2.3-1, and the force effects are divided by a multiple presence factor of 1.2 according to Article 3.6.1.1.2.

Interior portions of slab bridges designed using the equivalent strip width method are assumed to be adequate in shear (5.12.2.1), but edge beams on slab bridges require shear analysis. Provisions for edge beam equivalent strip widths and load distribution are given in Article 4.6.2.1.4b. The strip width for an edge beam is taken as the barrier width, plus 12 inches, plus one-quarter of the controlling strip width calculated for moment, not to exceed half the strip width calculated for moment or 72 inches.

For slab bridges with skewed supports, the force effects are reduced by a reduction factor r:

r = $1.05 - 0.25\tan\theta \leq 1.00$, where θ is the skew angle of the supports in degrees. (Eq. 4.6.2.3-3)

The live load distribution factor, with units "one lane, or two lines of wheels" per inch, is then taken as:

$$DF\ (\text{Single or Multiple Lanes Loaded}) = \frac{r}{E}$$

Or

$$DF\ (\text{Fatigue Truck Single Lane Loaded}) = \frac{r}{1.2E}$$

Determine Maximum Factored Moments

In analyzing main reinforcement for slab bridges, three load combinations are used:

Strength I load combination is defined as:

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

Where:

γ_p = For DC: maximum 1.25, minimum 0.90
For DW: maximum 1.50, minimum 0.65

Fatigue I load combination is defined as:

$$M_{FATIGUE\ I} = 1.75(LL+IM+CE) \qquad \text{(Table 3.4.1-1)}$$

For the Fatigue I load combination, all moments are calculated using the fatigue truck specified in Article 3.6.1.4. The fatigue truck is similar to the HL-93 truck, but with a constant 30 ft. rear axle spacing. Impact or dynamic load allowance is taken as 15% of the fatigue truck load for this load combination (Table 3.6.2.1-1).

Fatigue II load combination is not checked for slab bridges.

Service I load combination is defined as:

$M_{SERVICE\ I}$ = 1.0(DC+DW+LL+IM+CE) (Table 3.4.1-1)

For these load combinations, loads are abbreviated as follows:

DC = dead load of structural components (DC1) and non-structural attachments(DC2). This includes temporary concrete barriers used in stage construction.
DW = dead load of future wearing surface
LL = vehicular live load
IM = impact or dynamic load allowance
CE = vehicular centrifugal force, including forces due to bridge deck superelevation

Design Reinforcement in Slab

Main reinforcement in slab bridges is placed parallel to traffic except as allowed for some simple span skewed bridges. See Section 3.2.11 for the Bridge Manual for details. If possible, use the same size bars for all main reinforcement.

Four limit states are checked when designing main reinforcement: Flexural Resistance (5.6.3.2), Fatigue (5.5.3), Control of Cracking (5.6.7), and Minimum Reinforcement (5.6.3.3 & 5.5.4.2.1). These limit states should be checked at points of maximum stress and at theoretical cutoff points for reinforcement. See Figures 3.2.11-2 and 3.2.11-3 in the Bridge Manual for further guidance on determination of cutoff points for reinforcement. The deformation control parameters of Article 2.5.2.6 may be used in determining of slab thickness in the TSL phase, but are not mandatory requirements for final design.

Distribution reinforcement is not designed, but rather is a percentage of the main reinforcement. See All Bridge Designers Memorandum 15.8 for more details.

Check Flexural Resistance *(5.6.3.2)*

The factored resistance, M_r (k-in.), shall be taken as:

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

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

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

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 = $\frac{A_s f_s}{\alpha_1 \beta_1 f'_c\, b}$ (in.) (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 Fatigue (5.5.3)

For fatigue considerations, concrete members shall satisfy:

$\gamma(\Delta f) \leq (\Delta F)_{TH}$ (Eq. 5.5.3.1-1)

Where:

γ = load factor specified in Table 3.4.1-1 for the Fatigue I load combination
= 1.75

(Δf) = live load stress range due to fatigue truck (ksi)

$= \dfrac{\left| M^{+}_{FATIGUE\ I} - M^{-}_{FATIGUE\ I} \right|}{A_s j d_s}$

Designers should note that use of this formula neglects compression steel and assumes that behavior in areas of stress reversal will behave in compression in the same manner that they behave in tension. This is not an accurate assumption because concrete does not behave in tension in the same manner as it behaves in compression. However, this yields conservative results for steel stresses for slab bridges with ε_t > 0.005 (or ϕ = 0.9), and, unless compression steel were to be added to the design model, is the only way to consistently analyze the section.

$$(\Delta F)_{TH} = 26 - 22f_{min} / f_y \qquad \text{(Eq. 5.5.3.2-1)}$$

Where:

f_{min} = algebraic minimum stress level, tension positive, compression negative (ksi). The minimum stress is taken as that from unfactored factored dead loads (DC1 and DC2 with the inclusion of DW at the discretion of the designer), combined with that produced by $M^{-}_{FATIGUE\ I}$ in positive moment regions or $M^{+}_{FATIGUE\ I}$ in negative moment regions.

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)

$S = \frac{1}{6}bh^2$ (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

Design Distribution Reinforcement *(ABD 15.8, 5.10.6)*

Main distribution reinforcement is not designed, but rather is specified as a percentage of the main bottom reinforcement area. For slab bridges, the percentage is found using the following equations:

Simple Spans, Bottom Transverse Distribution Reinforcement

$A_{s(bot,trans)} = \beta_{total(bot)} \times A_{s(bot,long)}$

$\beta_{total(bot)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \le \beta_{max}$

where:

$\beta_{total(bot)}$ = factor of main bottom longitudinal reinforcement.

β_{base} = 0.21

β_{skew} = $\tan\theta \times 0.35\,(1 + 0.02(L - 20))$

β_{length} = $0.30 - 0.0075L \ge 0.0$

β_{width} = $0.02\sqrt{W - 24} \ge 0.0$

β_{max} = 0.70

L = span length according to Table 4.6.2.2.1-2 (ft.)

W = physical edge-to-edge final width of bridge (ft.)

θ = skew angle (degrees)

Simple Spans, Top Transverse and Top Longitudinal Reinforcement

$A_{s(top,trans)} = \beta_{total(top)} \times A_{s(bot,long)}$

$A_{s(top,long)} = \beta_{total(top)} \times A_{s(bot,long)}$

$\beta_{total(top)} = 0.20$

where:

$\beta_{total(top)}$ = factor of main bottom longitudinal reinforcement.

Continuous Spans, Bottom Transverse Distribution Reinforcement

$A_{s(bot,trans)} = \beta_{total(bot)} \times A_{s(bot,long)}$

$\beta_{total(bot)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \times 1.1 \leq \beta_{max}$

where:

$\beta_{total(bot)}$	=	factor of main bottom longitudinal reinforcement.
β_{base}	=	0.21
β_{skew}	=	$\tan\theta \times 0.2 (1 + 0.02(L - 20))$
β_{length}	=	$0.32 – 0.0055L \geq 0.0$
β_{width}	=	$0.02\sqrt{W - 24} \geq 0.0$
β_{max}	=	0.80
L	=	span length according to Table 4.6.2.2.1-2 (ft.)
W	=	physical edge-to-edge final width of bridge (ft.)
θ	=	skew angle (degrees)

Continuous Spans, Top Transverse Distribution Reinforcement

$A_{s(top,trans)} = \beta_{total(top)} \times A_{s(top,long)}$

$\beta_{total(top)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \times 1.2 \leq \beta_{max}$

where:

$\beta_{total(top)}$	=	factor of main top longitudinal reinforcement.
β_{base}	=	0.24
β_{skew}	=	$(\tan\theta)(0.55 (1 - 0.013(L - 20))) \geq 0.0$
β_{length}	=	$0.12 – 0.0025L \geq 0.0$
β_{width}	=	$(\sin\theta)\left(0.02\sqrt{W - 24}\right) (L / 20) \geq 0.0$
β_{max}	=	1.00
L	=	span length according Table 4.6.2.2.1-2 (ft.)
W	=	physical edge-to-edge final width of bridge (ft.)
θ	=	skew angle (degrees)

The required area of top distribution reinforcement, A_s, in square inches per foot width, should not be taken as less than that required for temperature and shrinkage:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \quad \text{(Eq. 5.10.6-1)}$$

Where h is the slab depth (in.), and b is the total width of the slab (in).

Spacing for reinforcement designed for shrinkage and temperature is not to be taken as greater than 18 inches center-to-center or three times the slab thickness.

Development of Reinforcement *(5.10.8)*

Provisions for development of reinforcement are found in Article 5.10.8. See also Figures 3.2.11-2 and 3.2.11-3 of the Bridge Manual for additional guidance on development lengths, detailing and bar cutoffs.

Edge Beams *(5.7.3.3)*

Interior strips for slab bridges designed using the distribution factors of 4.6.2.3 are considered adequate for shear (5.12.2.1). However, at the exteriors of bridges and along stage lines, edge beams must be designed. This involves calculating an edge beam width and separate edge beam live loading. The edge beam is then designed for shear, but the edge beam may control the flexural design as well. If this is the case, the edge beam is designed for both shear and flexure, and the longitudinal reinforcement of the edge beam is used for the entire width of the structure.

For many bridges, checking the design moment for the edge beam against the design moment for the interior strip is all that is necessary to verify that the edge beam does not control the flexural design. In the following design example, the edge beam does not control the flexural design, and calculations are performed to illustrate that.

Edge beams at stage construction lines must also be evaluated. This may require the addition of shear reinforcement in the slab and/or a concrete drop beam along the stage construction line.

Shear of concrete is checked at the critical section for shear. The critical section for shear is taken at a distance d_v from the face of the support (5.7.3.2). Because the longitudinal reinforcement is included in the shear design, and the longitudinal reinforcement may not be fully developed at the critical section for shear, a reinforcement development check is also required when shear is checked near abutments.

Shear Resistance (5.7.3.3)

The shear resistance of a concrete section is taken as the lesser of the following:

$$\phi V_n = \phi V_c + \phi V_s \quad \text{(Eq. 5.7.3.3-1)}$$

$$\phi V_n = \phi 0.25 f'_c b_v d_v \quad \text{(Eq. 5.7.3.3-2)}$$

The factored concrete shear resistance, ϕV_c (kips), is found using the following equation:

$$\phi V_c = \phi 0.0316 \beta \sqrt{f'_c}\, b_v d_v \quad \text{(Eq. 5.7.3.3-3)}$$

Where:

ϕ = 0.90 for shear (5.5.4.2)

b_v = strip width of edge beam (in.)

d_v = effective shear depth

= $d_s - \frac{a}{2}$, where d_s and a are as defined in flexural resistance calculations.

d_v need not be taken as less than the greater of $0.9d_s$ or $0.72h$ (in.)

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} \quad \text{(Eq. 5.7.3.4.2-1)}$$

$$\varepsilon_s = \frac{\frac{M_u}{d_v} + V_u}{E_s A_s} \qquad \text{(Eq. 5.7.3.4.2-4)}$$

M_u = Strength I moment applied at critical section for shear, not to be taken as less than $V_u d_v$

V_u = Strength I shear applied at critical section for shear

E_s = 29000 ksi (6.4.1)

A_s = area of longitudinal steel on flexural tension side of member (in.^{2})

At edges of slabs with F-Shape parapets and standard IDOT reinforcement, the factored shear steel resistance, ϕV_s is found using the following equation:

$$\phi V_s = \phi \frac{A_v f_y d_v (\cot\theta + \cot\alpha)(\sin\alpha)}{s} \qquad \text{(Eq. 5.7.3.3-4)}$$

Where:

ϕ = 0.90 for shear (5.5.4.2)

s = spacing of stirrups (in)

A_v = area of shear reinforcement within a distance s (in.^{2})

d_v = effective shear depth

= $d_s - \frac{a}{2}$, where d_s and a are as defined in flexural resistance calculations.

d_v need not be taken as less than the greater of $0.9d_s$ or 0.72h (in.)

θ = angle of inclination of stresses

= $29 + 3500\varepsilon_s$, where ε_s is as calculated above

α = inclination of reinforcement, taken as 90 degrees

f_y = yield strength of steel (ksi)

When necessary, an edge beam may be thickened at an open joint using a concrete haunch until shear capacity is met.

LRFD Slab Bridge Design Example: Two-Span Slab Bridge, 30 degree Skew

Design Stresses

f'_c = 4 ksi

f_y = 60 ksi

Bridge Data

Span Lengths:	Two 36 ft. Spans
Bridge Slab Width:	32 ft. Out-To-Out Including F-Shape Parapets
Slab Thickness:	16 in.
Future Wearing Surface:	50 psf
Skew Angle θ:	30 degrees

Note: Design at theoretical cutoff points not included in this example.

Determine Dead Load Unit Weights

Slab: $\left(0.15\ \text{k/ft.}^3\right)\left(\dfrac{16\ \text{in.}}{12\ \text{in./ft.}}\right)$ = 0.200 k/ft.

Parapets: $\dfrac{(0.45\ \text{k/ft.})(2\ \text{parapets})}{32\ \text{ft.}}$ = 0.028 k/ft.

FWS: $\left(0.050\ \text{k/ft.}^2\right)(1\ \text{ft.})$ = 0.050 k/ft.

Determine Live Load Distribution Factors (LLDFs) *(4.6.2.3)*

<u>Skew Reduction Factor r</u>

r = $1.05 - 0.25\tan\theta \leq 1.00$

= $1.05 - 0.25 \tan 30$

= 0.905

<u>For Multiple-Lanes Loaded</u>

$$E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \frac{12.0W}{N_L}$$ (Eq. 4.6.2.3-2)

Where:

N_L = 2 lanes

W = 32 ft.

L_1 = 36 ft.

W_1 = min(bridge width, 60 ft.) for multiple lanes loaded

= 32 ft.

$$84.0 + 1.44\sqrt{L_1 W_1} = 84.0 + 1.44\sqrt{(36 \text{ ft.})(32 \text{ ft.})} = 132.88 \ \frac{\text{in.}}{\text{lane}}$$

$$\frac{12.0W}{N_L} = \frac{12.0(32)}{2} = 192.00 \ \frac{\text{in.}}{\text{lane}}$$

$$\therefore \ E = 132.88 \ \frac{\text{in.}}{\text{lane}}$$

$$\text{LLDF} = 0.905 \frac{1 \text{ lane}}{132.88 \text{ in.}}\left(\frac{12 \text{ in.}}{\text{ft.}}\right) = 0.0818 \frac{\text{lanes}}{\text{ft. of slab width}}$$

For Single-Lane Loaded

$$E = 10.0 + 5.0\sqrt{L_1 W_1} \qquad \text{(Eq. 4.6.2.3-1)}$$

Where:

L_1 = 36 ft.

W_1 = min(bridge width, 30 ft.) for single lane loaded

= 30 ft.

$$E = 10.0 + 5.0\sqrt{(36 \text{ ft.})(30 \text{ ft.})} = 174.32 \ \frac{\text{in.}}{\text{lane}}$$

$$\text{LLDF} = 0.905 \frac{1 \text{ lane}}{174.32 \text{ in.}}\left(\frac{12 \text{ in.}}{\text{ft.}}\right) = 0.0622 \frac{\text{lanes}}{\text{ft. of slab width}}$$

0.0622 < 0.0818, therefore the LLDF for multiple lanes loaded, 0.0818, controls.

For Fatigue Loading

The LLDF for Fatigue loading is the single lane loaded LLDF divided by a multiple presence factor of 1.2.

$$\text{LLDF (Modified for Fatigue)} = \frac{0.0622}{1.2} = 0.052\ \frac{\text{lanes}}{\text{ft. of slab width}}$$

<u>Edge Beam Width</u> <u>(4.6.2.1.4b)</u>

The design edge beam width, E, for 1-line of wheel loading (½ lane) shall be taken as:

Width of parapet base + 12 in. + ¼ of strip width for a single lane loaded, not to exceed ½ the full strip width or 72 in.

$E = 19\text{ in} + 12\text{ in.} + \frac{1}{4}(174.3\text{ in.}) = 74.5\text{ in.} > 72\text{ in.}$

$\therefore E = 72\text{ in.}$

Per AASHTO, the design shear should account for all dead loads within this width, half of one truck or tandem, and the tributary lane load. The tributary lane load width will be the width of the edge beam that can accommodate a traffic lane, or, in other words, the portion of the edge beam that is more than one foot from the parapet. This is the total edge beam width minus the parapet base width minus twelve inches:

$$\text{Fraction of lane load} = \frac{E - \text{parapet base width}}{10\text{ ft.}(12\text{ in./ft.})}$$

$$= \frac{72\text{ in.} - 19\text{ in.}}{10\text{ ft.}(12\text{ in./ft.})}$$

$$= 0.44$$

Determine Maximum Factored Moments

At point 0.4L in span 1 (near the point of maximum positive moment) and 0.6L in span 2, the unfactored distributed moments are:

$M_{DC1} = 18.1$ k-ft.

M_{DC2} = 2.3 k-ft.

M_{DW} = 4.5 k-ft.

M_{Truck} = 317.8 k-ft.

M_{Tandem} = 343.3 k-ft.

M_{Lane} = 58.1 k-ft.

The tandem load controls over the truck load. The two-truck loading was found to not control. The impact factor is 1.33, applied to the tandem only. The controlling unfactored live load plus impact (LL+IM) moment is therefore the larger of either:

LL+IM on the interior strip = $LLDF(1.33M_{Tandem} + M_{Lane})$

= 0.0818(1.33(343.3 k-ft.) + 58.1 k-ft.)

= 42.1 k-ft.

LL+IM on the edge beam $= \dfrac{1.33(0.5)(M_{Tandem}) + \text{Fraction of Lane Load}(M_{Lane})}{\text{Edge Beam Width}}$

$= \dfrac{1.33(0.5)(343.3\ \text{k} - \text{ft.}) + (0.44)(58.1\ \text{k} - \text{ft.})}{72\ \text{in.}(1\ \text{ft.}/12\ \text{in.})}$

= 42.3 k-ft.

The edge beam controls the design at 0.4L. The factored Strength I and Service I loads are then:

$M_{STRENGTH\ I}$ = 1.25(18.1 k-ft. + 2.3 k-ft.) + 1.5(4.5 k-ft.) + 1.75(42.3 k-ft.)

= 106.3 k-ft. per foot width

$M_{SERVICE\ I}$ = 1.00(18.1 k-ft. + 2.3 k-ft.) + 1.00(4.5 k-ft.) + 1.00(42.3 k-ft.)

= 67.2 k-ft. per foot width

At 0.4L, the fatigue truck moments were found to be 15.2 k-ft. maximum and -2.9 k-ft. minimum.

At point 1.0L in span 1 (the point of maximum negative moment), the unfactored distributed moments are:

M_{DC1} = -32.4 k-ft.

M_{DC2} = -4.1 k-ft.

M_{DW} = -8.1 k-ft.

M_{Truck} = -236.3 k-ft.

M_{Tandem} = -170.5 k-ft.

M_{Lane} = -103.7 k-ft.

Note that use of the maximum negative moment values is conservative. As per 5.6.3 of the AASHTO LRFD Bridge Design Specifications, design moments may be taken at the face of the pier for flexural checks. Moments at the face of the pier will be less extreme than those at the centerline of the pier due to the shape of the moment diagram.

The truck load controls over the tandem load. The two-truck loading was found to not control. The impact factor is 1.33, applied to the truck only. The controlling unfactored live load plus impact (LL+IM) moment is therefore the larger of either:

LL+IM on the interior strip = $LLDF(1.33M_{Truck} + M_{Lane})$

= 0.0818(1.33(-236.3 k-ft.) + -103.7 k-ft.)

= -34.2 k-ft.

$$\text{LL+IM on the edge beam} = \frac{1.33(0.5)(M_{Truck}) + \text{Fraction of Lane Load}(M_{Lane})}{\text{Edge Beam Width}}$$

$$= \frac{1.33(0.5)(-236.3\text{ k}-\text{ft.}) + (0.44)(-103.7\text{ k}-\text{ft.})}{72\text{ in.}(1\text{ ft.}/12\text{ in.})}$$

= -33.8 k-ft.

The interior strip controls the design at 1.0L. The factored Strength I and Service I loads are then:

$M_{STRENGTH\ I}$ = 1.25(-32.4 k-ft. + -4.1 k-ft.) + 1.5(-8.1 k-ft.) + 1.75(-34.2 k-ft.)

= -117.6 k-ft. per foot width

$M_{SERVICE\ I}$ = 1.00(-32.4 k-ft. + -4.1 k-ft.) + 1.00(-8.1 k-ft.) + 1.00(-34.2 k-ft.)

= -78.8 k-ft. per foot width

At 1.0L, the fatigue truck moments were found to be 0.0 k-ft. maximum and -14.1 k-ft. minimum.

At point 0.93L in span 1 (the critical section for shear) the unfactored undistributed moments and shears per unit width are as follows. DC and DW values are shown for a one foot strip width. Live load values are for an entire lane.

M_{DC1}	=	-21.8 k-ft. / ft.	V_{DC1}	=	-4.0 k / ft.
M_{DC2}	=	-2.7 k-ft. / ft.	V_{DC2}	=	-0.5 k / ft.
M_{DW}	=	-5.4 k-ft. / ft.	V_{DW}	=	-1.0 k / ft.
M_{Truck}	=	-237.4 k-ft.	V_{Truck}	=	-52.7 k
M_{Tandem}	=	-213.1 k-ft.	V_{Tandem}	=	-46.3 k
M_{Lane}	=	-69.7 k-ft.	V_{Lane}	=	-12.7 k

The truck load controls over the tandem load. The two-truck loading was found to not control. The impact factor is 1.33, applied to the truck only. Because shear is not checked for interior strips, only the edge beam moments and shears need be calculated.

The LL+IM shear and moment are calculated for the entire 6 ft. beam width.

The unfactored LL+IM moment is:

$$\text{LL+IM on the edge beam} = 1.33(0.5)(M_{Truck}) + \text{Fraction of Lane Load}(M_{Lane})$$

$$= 1.33(0.5)(-237.4\ \text{k}-\text{ft.}) + (0.44)(-69.7\ \text{k}-\text{ft.})$$

$$= -188.5\ \text{k-ft.}$$

The factored Strength I moment for the edge beam is then:

$$M_{STRENGTH\ I} = 1.25(-21.8\ \text{k-ft. / ft.} + -2.7\ \text{k-ft. / ft.})(6\ \text{ft.}) + 1.5(-5.4\ \text{k-ft. / ft.})(6\ \text{ft.}) + 1.75(-188.5\ \text{k-ft.})$$

$$= -562.2\ \text{k-ft. for the entire six-foot edge beam width}$$

The unfactored LL+IM shear is:

LL+IM on the edge beam $= 1.33(0.5)(V_{Truck}) + \text{Fraction of Lane Load}(V_{Lane})$

$= 1.33(0.5)(-52.7\text{ k}) + (0.44)(-12.7\text{ k})$

$= -40.6\text{ k}$

The factored Strength I shear for the edge beam is then:

$V_{STRENGTH\ I}$ = 1.25(-4.0 k / ft. + -0.5 k / ft.)(6 ft.) + 1.5(-1.0 k / ft.)(6 ft.) + 1.75(-40.6 k)

= -113.8 k for the entire six-foot edge beam width

Design Positive Moment Reinforcement

<u>Check Flexural Resistance @ 0.4 Span 1</u> <u>(5.6.3.2)</u>

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

Assume #9 bars, solve for A_s:

b = 12 in.

d_s = 16 in. – 1.5 in. clear – 0.5(1.128 in. bar diameter) = 13.94 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

α_1 = 0.85 (5.6.2.2)

β_1 = 0.85 (5.6.2.2)

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

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

$$M_r = M^+_{STRENGTH\ I} = 106.3\text{ k-ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right) = 1275.6\text{ k-in.}$$

$$1275.6\text{ k-in.} = (0.9)\left[A_s(60\text{ ksi})\left(13.94\text{ in.} - \frac{1.47A_s\text{ in.}}{2}\right)\right]$$

Solving for A_s gives A_s = 1.88 in.^{2} Try #9 bars @ 6 in. center-to-center spacing, A_s = 2.00 in.^{2}

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

$c = 1.73\,A_s = 1.73(2.00\text{ in.}^2) = 3.46\text{ in.}$

$d_s = 13.94\text{ in.}$

$$\frac{c}{d_s} = \frac{3.46\text{ in.}}{13.94\text{ in.}} = 0.25 < 0.6 \quad \therefore \text{Assumption of } f_s = f_y = 60\text{ ksi is valid.}$$

Verify ϕ = 0.9 assumption:

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

Where:

$c = 3.46\text{ in.}$

$d_t = d_s = 13.94\text{ in.}$

$$\varepsilon_t = \frac{0.003(13.94\text{ in.} - 3.46\text{ in.})}{3.46\text{ in.}} = 0.009$$

0.009 > 0.005, $\therefore$ Assumption of ϕ = 0.9 is valid.

Check Control of Cracking @ 0.4 Span 1 (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 = 1.5 in. clear + 0.5(1.128 in. bar diameter) = 2.064 in.

h = 16 in.

$$\beta_s = 1 + \frac{2.064\text{ in.}}{0.7(16\text{ in.} - 2.064\text{ in.})} = 1.212$$

$$\rho = \frac{2.00\ \text{in.}^2}{(12\ \text{in.})(13.94\ \text{in.})} = 0.0120$$

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

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

$$k = \sqrt{[(0.0120)(7.27)]^2 + 2(0.0120)(7.27)} - (0.0120)(7.27) = 0.339$$

$$j = 1 - \frac{0.339}{3} = 0.887$$

$$f_{ss} = \frac{(67.0\ \text{k}-\text{ft.})\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{(2.00\ \text{in.}^2)(0.887)(13.94\ \text{in.})} = 32.51\ \text{ksi} < 06f_y = 36\ \text{ksi}\quad \text{O.K.}$$

γ_e = 0.75

$$\frac{700\gamma_e}{\beta_s f_s} - 2d_c = \frac{700(0.75)}{(1.212)(32.51)} - 2(2.064) = 9.2\ \text{in.}$$

s = 6 in. < 11.4 in. O.K.

∴ #9 bars @ 6 in. center-to-center spacing is adequate to control cracking.

Check Fatigue @ 0.4 Span 1 (5.5.3)

For other structures fatigue may be more critical at a different location. The requirements of 5.5.3 should be satisfied throughout the span length.

$\gamma(\Delta f) \leq (\Delta F)_{TH}$ (5.5.3.1-1)

Where:

$$\gamma(\Delta f) = \frac{\left|M^{+}_{FATIGUE\ I} - M^{-}_{FATIGUE\ I}\right|}{A_s j d_s}$$

$$M^{+}_{FATIGUE\ I} = 1.75(15.2\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) = 319.2\text{ k-in.}$$

$$M^{-}_{FATIGUE\ I} = 1.75(-2.9\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) = -60.9\text{ k-in.}$$

$$\gamma(\Delta f) = \frac{\left|(319.2\text{ k}-\text{in.}) - (-60.9\text{ k}-\text{in.})\right|}{(2.00\text{ in.}^2)(0.889)\ (13.94\text{ in.})} = 15.33\text{ ksi}$$

$$(\Delta F)_{TH} = 26 - 22f_{min}/f_y \qquad \text{(Eq. 5.5.3.2-1)}$$

The minimum live Fatigue I loading is -60.9 k-in.

Unfactored dead load moment is 18.3 k-ft. + 2.3 k-ft. + 4.5 k-ft. = 24.9 k-ft.

$$f_{min} = \frac{(-60.9\text{ k}-\text{in.}) + (24.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{(2.00\text{ in.}^2)(0.887)(13.94\text{ in.})} = 9.6\text{ ksi}$$

$26 - 22f_{min}/f_y$ = 26 – 22(9.6)/(60) = 22.5 ksi

15.33 ksi < 22.6 ksi O.K.

$\therefore$#9 bars @ 6 in. center-to-center spacing is adequate for fatigue limit state.

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

$$S = \frac{1}{6}(12\text{ in.})(16\text{ in.})^2 = 512\text{ in.}^3$$

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

M_{cr} = 0.75(1.6)(512 in.^{3})(0.48 ksi) = 294.9 k-in.

$1.33M_{STRENGTH\ I}$ = 1.33(1275.6 k-in.) = 1696.5 k-in

min(M_{cr}, $1.33M_{STRENGTH\ I}$) = 294.9 k-in.

$$\phi M_n = 0.9\left(2.00\text{ in.}^2\right)\left(60\text{ ksi}\right)\left[13.94\text{ in.} - \frac{\left(2.00\text{ in.}^2\right)\left(60\text{ ksi}\right)}{2\left(0.85\right)\left(12\text{ in.}\right)\left(4\text{ ksi}\right)}\right] = 1346.7\text{k-in.}$$

1346.7 k-in. > 249.9 k-in. O.K.

Design Bottom Distribution Reinforcement *(ABD Memo 15.8, 5.10.6)*

<u>Continuous Spans, Bottom Transverse Distribution Reinforcement</u>

$A_{s(bot,trans)} = \beta_{total(bot)} \times A_{s(bot,long)}$

$\beta_{total(bot)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \times 1.1 \le \beta_{max}$

where:

β_{base} = 0.21

β_{skew} = tan 30 degrees x 0.2 (1 + 0.02(36 ft. – 20 ft.))

= 0.15

β_{length} = 0.32 – 0.0055(36 ft.)

= 0.12

β_{width} = $0.02\sqrt{32\text{ ft.} - 24\text{ ft.}}$

= 0.06

β_{max} = 0.80

$\beta_{total(bot)}$ = (0.21 + 0.15 + 0.12 + 0.06) x 1.1

= 0.59

$A_{s(bot,trans)}$ = 0.59 x 2.00 in.^{2}/ft

= 1.19 in.^{2}/ft

<u>Check Shrinkage and Temperature Requirements</u> <u>(5.10.6)</u>

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad \text{(Eq. 5.10.6-1)}$$

$$\geq \frac{1.30((32\text{ ft.})(12\text{ in./ft.}))(16\text{ in.})}{2(32\text{ ft.}(12\text{ in./ft.})+16\text{ in.})(60\text{ ksi})}$$

1.19 in.^{2}/ft > 0.17 in.^{2}/ft O.K - Distribution Reinforcement Controls

Try #7 bars at 6" cts., A_s = 1.20 in^2/ft

Design Negative Moment Reinforcement

Check Flexural Resistance @ Pier (5.6.3.2)

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

Assume #9 bars, solve for A_s:

- b = 12 in.
- d_s = 16 in. – 2.5 in. clear – 0.5(1.128 in. bar diameter) = 12.94 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
- α_1 = 0.85 (5.6.2.2)
- 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)
- $a = c\beta_1 = 0.85(1.73A_s) = 1.47A_s$ in.
- $M_r = M^{+}_{STRENGTH\ I} = 117.6\text{ k-ft.}\left(\dfrac{12\text{ in.}}{\text{ft.}}\right) = 1411.2\text{ k-in.}$

$$1411.2\text{ k-in.} = (0.9)\left[A_s(60\text{ ksi})\left(12.94\text{ in.} - \frac{1.47A_s\text{ in.}}{2}\right)\right]$$

Solving for A_s gives A_s = 2.32 in.^{2} Try #9 bars @ 5 in. center-to-center spacing, A_s = 2.40 in.^{2}

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

$$c = 1.73\,A_s = 1.73(2.40\text{ in.}^2) = 4.15\text{ in.}$$

$$d_s = 12.94\text{ in.}$$

$$\frac{c}{d_s} = \frac{4.15\text{ in.}}{12.94\text{ in.}} = 0.32 < 0.6 \quad \therefore \text{Assumption of } f_s = f_y = 60\text{ ksi is valid.}$$

Verify ϕ = 0.9 assumption:

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

Where:

c = 4.15 in.

$d_t = d_s$ = 12.94 in.

$$\varepsilon_t = \frac{0.003(12.94\text{ in.} - 4.15\text{ in.})}{4.15\text{ in.}} = 0.0064$$

0.0064 > 0.005, $\therefore$ Assumption of ϕ = 0.9 is valid.

Check Control of Cracking @ Pier (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(1.128 in. bar diameter) = 3.064 in.

h = 16 in.

$$\beta_s = 1 + \frac{3.064\text{ in.}}{0.7(16\text{ in.} - 3.064\text{ in.})} = 1.34$$

$$\rho = \frac{2.40\text{ in.}^2}{(12\text{ in.})(12.94\text{ in.})} = 0.0155$$

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

E_s = 29000 ksi (6.4.1)

E_c = $120000 K_1 w_c^2 f'_c{}^{0.33}$ (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

$$k = \sqrt{[(0.0155)(7.27)]^2 + 2(0.0155)(7.27)} - (0.0155)(7.27) = 0.376$$

$$j = 1 - \frac{0.376}{3} = 0.875$$

$$f_{ss} = \frac{(78.8\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{(2.40\text{ in.}^2)(0.875)(12.94\text{ in.})} = 34.8\text{ ksi} < 0.6f_y = 36\text{ ksi}$$ O.K.

γ_e = 0.75

$$\frac{700\gamma_e}{\beta_s f_s} - 2d_c = \frac{700(0.75)}{(1.34)(34.8)} - 2(3.064) = 5.13\text{ in.}$$

s = 5 in. < 5.13 in. O.K.

$\therefore$#9 bars @ 5 in. center-to-center spacing is adequate to control cracking.

Check Fatigue @ Pier (5.5.3)

$\gamma(\Delta f) \le (\Delta F)_{TH}$ (5.5.3.1-1)

Where:

$$\gamma(\Delta f) = \frac{\left|M^+_{FATIGUE\ I} - M^-_{FATIGUE\ I}\right|}{A_s j d_s}$$

$$M^{+}_{FATIGUE\ I} = 1.75(0\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) = 0\text{ k-in.}$$

$$M^{-}_{FATIGUE\ I} = 1.75(-14.1\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) = -296.1\text{ k-in.}$$

$$\gamma(\Delta f) = \frac{\left|(0\text{ k}-\text{in.})-(-296.1\text{ k}-\text{in.})\right|}{(2.40\text{ in.}^2)(0.875)(12.94\text{ in.})} = 10.90\text{ ksi}$$

$$(\Delta F)_{TH} = 26 - 22f_{min}/f_y \qquad \text{(Eq. 5.5.3.2-1)}$$

The Service I dead load moment is 32.4 k-ft. + 4.1 k-ft. + 8.1 k-ft. = 44.6 k-ft.

$$f_{min} = \frac{(0\text{ k}-\text{ft.}+44.6\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{(2.40\text{ in.}^2)(0.875)(12.94\text{ in.})} = 19.70\text{ ksi}$$

$$26 - 22f_{min}/f_y = 26 - 22(19.70)/(60) = 18.77\text{ ksi}$$

10.90 ksi < 18.77 ksi O.K.

$\therefore$#9 bars @ 5 in. center-to-center spacing is adequate for fatigue limit state.

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

$S = \frac{1}{6}(12\text{ in.})(16\text{ in.})^2 = 512\text{ in.}^3$

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

$M_{cr} = 0.75(1.6)(512\text{ in.}^3)(0.48\text{ ksi}) = 294.9\text{ k-in.}$

$1.33M_{STRENGTH\ I} = 1.33(1411.2\text{ k-in.}) = 1876.9\text{ k-in.}$

$\min(M_{cr}, 1.33M_{STRENGTH\ I}) = 294.9$ k-in.

$$\phi M_n = 0.9\left(2.40\ \text{in.}^2\right)\left(60\ \text{ksi}\right)\left[12.94\ \text{in.} - \frac{\left(2.40\ \text{in.}^2\right)\left(60\ \text{ksi}\right)}{2\left(0.85\right)\left(12\ \text{in.}\right)\left(4\ \text{ksi}\right)}\right] = 1447.6\ \text{k-in.}$$

1447.6 k-in. > 294.9 k-in. O.K.

Design Top Distribution Reinforcement (5.14.4.1)

Continuous Spans, Top Transverse Distribution Reinforcement

$A_{s(top,trans)} = \beta_{total(top)} \times A_{s(top,long)}$

$\beta_{total(top)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \times 1.2 \leq \beta_{max}$

where:

β_{base} = 0.24

β_{skew} = tan 30 degrees x 0.55 (1 - 0.013(36 ft. – 20 ft.))

= 0.25

β_{length} = 0.12 – 0.0025(36 ft.)

= 0.03

β_{width} = (sin 30 degrees)($0.02\sqrt{32\ \text{ft.} - 24\ \text{ft.}}$)(36 ft. / 20 ft.)

= 0.05

β_{max} = 1.00

$\beta_{total(top)}$ = (0.24 + 0.25 + 0.03 + 0.05) x 1.2

= 0.68

$A_{s(top,trans)}$ = 0.68 x 2.40 in.^{2}/ft

= 1.63 in.^{2}/ft

Check Shrinkage and Temperature Requirements (5.10.6)

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$ (Eq. 5.10.6-1)

$$\geq \frac{1.30((32\text{ ft.})(12\text{ in./ft.}))(16\text{ in.})}{2(32\text{ ft.}(12\text{ in./ft.})+16\text{ in.})(60\text{ ksi})}$$

1.63 in.^{2}/ft > 0.17 in.^{2}/ft O.K. – Distribution Reinforcement Controls

Try #9 bars at 7" cts., A_s = 1.71 in^2/ft.

Verify Edge Beam Adequacy

Determine Shear Resistance @ 0.93 Span 1 (5.7.3.3)

The shear resistance of a concrete section is taken as the lesser of the following:

$\phi V_n = \phi V_c + \phi V_s$ (Eq. 5.7.3.3-1)

$\phi V_n = \phi 0.25 f'_c b_v d_v$ (Eq. 5.7.3.3-2)

The factored concrete shear resistance, ϕV_c (kips), is found using the following equation:

$$\phi V_c = \phi 0.0316 \lambda \beta \sqrt{f'_c}\, b_v d_v$$ (Eq. 5.7.3.3-3)

Where:

ϕ = 0.90 for shear (5.5.4.2)

b_v = 72 in.

$$d_v = d_s - \frac{a}{2}$$

$$= 12.94\text{ in.} - \frac{1.47(2.40\text{ in.}^2)}{2}$$

= 11.18 in.

$0.72h$ = 0.72(16 in.)

= 11.52 in.

$0.9d_e$ = 0.9(12.94 in.)

= 11.65 in., which controls

d_v = 11.65 in.

$$\varepsilon_s = \frac{\dfrac{(631.2\ \text{k}-\text{ft.})(12\ \text{in./ft.})}{11.65\ \text{in.}} + 113.5\ \text{k}}{(29000\ \text{ksi})(2.4\ \text{in.}^2/\text{ft.})(72\ \text{in.})(1\ \text{ft.}/12\ \text{in.})}$$ (Eq. 5.7.3.4.2-4)

$= 0.00183$

$$\beta = \frac{4.8}{1 + 750(0.00183)}$$ (Eq. 5.7.3.4.2-2)

$= 2.02$

λ = 1.0 for normal-weight concrete

$$\phi V_c = 0.9(0.0316)(2.02)\sqrt{4\ \text{ksi}}(72\ \text{in.})(11.65\ \text{in.}) = 96.4\ \text{k}$$

$$\phi V_s = \phi \frac{A_v f_y d_v (\cot\theta + \cot\alpha)(\sin\alpha)}{s}$$ (Eq. 5.7.3.3-4)

Where:

ϕ = 0.90 for shear (5.5.4.2)

s = 11 in.

A_v = 0.62 in.^{2}

d_v = 11.65 in.

θ = 29 + 3500(0.00183)

= 35.41 degrees

α = 90 degrees

f_y = 60 ksi

$$\phi V_s = (0.9)\frac{(0.62\ \text{in.}^2)(60\ \text{ksi.})(11.65\ \text{in.})(\cot 35.41^\circ + \cot 90^\circ)(\sin 90^\circ)}{11\ \text{in.}}$$

$= 49.8$ k

$\phi V_c + \phi V_s$ = 96.4 k + 49.8 k = 146.2 k > 111.7 k

Check $\phi V_n = \phi 0.25 f'_c b_v d_v$

$\phi 0.25 f'_c b_v d_v$ = (0.9)(0.25)(4 ksi)(72 in.)(11.64 in.)

= 754.3 k > 111.7 k O.K.