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18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison to the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures.

18.1.2 Limitations

Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet; formwork removal requires this clearance. When spans exceed 35 feet, freeboard shall be increased to 5 feet above normal water.

All concrete slab structures are limited to a maximum skew of 30 degrees. Slab structures with skews in excess of 30 degrees, require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bureau of Structures.

Continuous span slabs are to be designed using the following pier types:

- Piers with pier caps (on columns or shafts)
- Wall type piers

These types will allow for ease of future superstructure replacement. Piers that have columns without pier caps, have had the columns damaged during superstructure removal. This type of pier will not be allowed without the approval of the Structures Design Section.

WisDOT policy item:

Slab bridges, due to camber required to address future creep deflection, do not ride ideally for the first few years of their service life and present potential issues due to ponding. As such, if practical (e.g. not excessive financial implications), consideration of other structure types should be given for higher volume/higher speed facilities, such as the Interstate. Understanding these issues, the Regions have the responsibility to make the final decision on structure type with respect to overall project cost, with BOS available for consultation.

18.2 Specifications, Material Properties and Structure Type

18.2.1 Specifications

Reference may be made to the design and construction related material as presented in the following specifications:

- *State of Wisconsin, Department of Transportation Standard Specifications for Highway and Structure Construction*

  Section 502 - Concrete Bridges

  Section 505 - Steel Reinforcement

- Other Specifications as referenced in Chapter 3

18.2.2 Material Properties

The properties of materials used for concrete slab structures are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

4 ksi, for concrete slab superstructure

3.5 ksi, for concrete substructure units

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in slab **LRFD [C5.4.2.4]**

= $33{,}000\ K_1\ w_c^{1.5}\ (f'_c)^{1/2} = 3800$ ksi

Where:

K_1 = 1.0

w_c = 0.150 kcf, unit weight of concrete

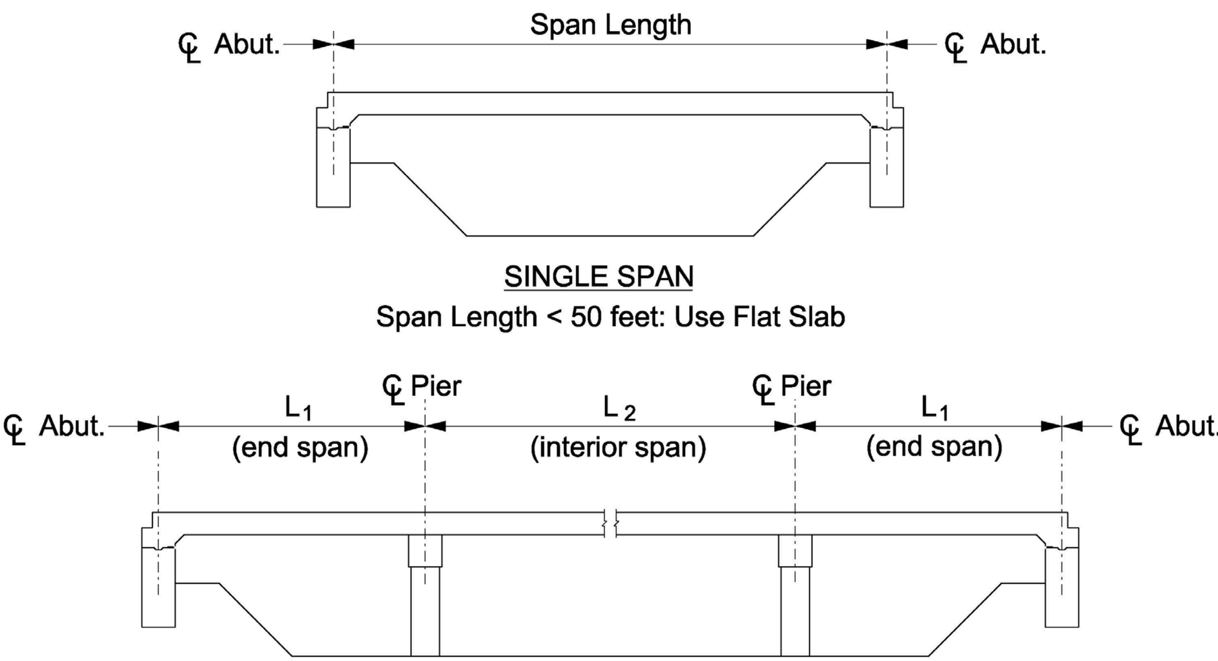
n = $E_s / E_c = 8$ **LRFD [5.6.1]** (modular ratio)

18.2.3 Structure Type and Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, approximate slab depth, skew, roadway width, etc.. The selection of the type of concrete slab structure (haunched / flat) is a function of the span lengths selected. Recommended span length ranges and corresponding structure type are shown for single-span and multiple-span slabs in Figure 18.2-1. Estimated slab depths are shown in Table 18.2-1.

Currently, voided slab structures are not allowed. Some of the existing voided slabs have displayed excessive longitudinal cracking over the voids in the negative zone. This may have been caused by the voids deforming or floating-up due to lateral pressure during the concrete pour. Recent research indicates slabs with steel void-formers have large crack widths above the voids due to higher stress concentrations.

If optimum span ratios are selected such that the positive moments in each span are equal, the interior and end span slab depths will be equal, provided Strength Limit State controls. Optimum span ratios are independent of applied live loading.



TWO OR MORE SPANS

All Span Lengths < 35 feet $\pm$: Use Flat Slab Throughout
Any Span Length $\geq$ 45 feet $\pm$ < 70 feet: Use Haunched Slab Throughout
Other Spans: Consider Economics, Aesthetics and Clearances

Figure 18.2-1
Span Length vs. Slab Type

For the following optimum span ratio equations based on Strength Limit State controlling, L_1 equals the end span lengths and L_2 equals the interior span length or lengths, for structures with three or more spans.

For flat slabs the optimum span ratio is obtained when $L_2 = 1.25L_1$. The optimum span ratio for a three-span haunched slab results when $L_2 = L_1(1.43 - 0.002L_1)$ and for a four-span haunched slab when $L_2 = 1.39L_1$.

Approximate slab depths for multiple-span flat and haunched slabs can be obtained from Table 18.2-1. These values are to be used for dead load computations and preliminary computations only and the final slab depth is to be determined by the designer.

(s) Span Length (feet)	Slab Depth (inches)	
	Haunched [1]	Flat [4]
20	---	12
25	---	14
30	---	16
35	---	18
40	---	20
45	16 [2]	22
50	17.5 [2]	24
55	19 [2]	26
60	20 [2]	---
65	22 [3]	---
70	25 [3]	---

Table 18.2-1
Span Length vs. Slab Depth

[1] These estimated slab depths at mid-span, apply to interior spans of three or more span structures, with an end span length of approximately 0.7 times the interior span. Depths are based on dead load deflection (camber) and live load deflection limits. Haunch length (L_{haunch}) = 0.167 (L_2), and d_{slab} / D_{haunch} = 0.6 were used. L_2 = interior span length, (d_{slab}) = slab depth in span and (D_{haunch}) = slab depth at haunch. Values in table include ½ inch wearing surface.

[2] Depths controlled by live load deflection criteria

[3] Depths controlled by dead load deflection (camber) criteria

[4] These values represent **LRFD [2.5.2.6.3]** recommended minimum depths for continuous-spans using (s+10)/30. The slab span length (s) in the equation and resulting minimum depths are in feet and are presented in inches in Table 18.2-1. For simple-spans, the Bureau of Structures adds 10% greater depth and checks the criteria in 18.4.4. Values in table include ½ inch wearing surface.

The minimum slab depth is 12 inches. Use increments of ½ inch to select depths > 12 inches.

18.3 Limit States Design Method

18.3.1 Design and Rating Requirements

All new concrete slab structures are to meet design requirements as stated in 17.1.1 and rating requirements as stated in 17.1.2.

18.3.2 LRFD Requirements

18.3.2.1 General

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: **LRFD [1.3.2.1, 5.5.1]**

$$Q = \sum \eta_i \, \gamma_i \, Q_i \leq \phi \, R_n = R_r$$ (Limit States Equation) **LRFD [1.3.2.1, 3.4.1]**

Where:

η_i	=	load modifier (a function of η_D , η_R and η_I) **LRFD [1.3.2.1, 1.3.3, 1.3.4, 1.3.5]**
γ_i	=	load factor
Q_i	=	force effect; moment, shear, stress range or deformation caused by applied loads
Q	=	total factored force effect
ϕ	=	resistance factor
R_n	=	nominal resistance; resistance of a component to force effects
R_r	=	factored resistance = $\phi \, R_n$

The Limit States used for concrete slab design are:

- Strength I Limit State
- Service I Limit State
- Fatigue I Limit State

18.3.2.2 Statewide Policy

Current Bureau of Structures policy is :

- Set value of load modifier, η_i , and its factors (η_D , η_R , η_I) all equal to 1.00 for concrete slab design.

- Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects, Q_i , for slab bridges.

- Ignore reduction factor, r, for skewed slab bridges in **LRFD [4.6.2.3]** that would reduce longitudinal force effects, Q_i .

18.3.3 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life **LRFD [1.3.2.4]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Strength I Limit State **LRFD [3.4.1]** will be used for:

- Designing longitudinal slab reinforcement for flexure

- Designing transverse slab reinforcement over the piers for flexure

- Checking shear (two-way) in slab at the piers

- Checking uplift at the abutments

- Checking longitudinal slab reinforcement for tension from shear

18.3.3.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Strength I Limit State will be used to design the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, defined in 18.4.3.2.

The load factor, γ_i , is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

For Strength I Limit State, the values of γ_i for each applied load, are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]** and their values are: γ_{DC} = 1.25/0.90, γ_{DW} = 1.50/0.65, γ_{LL+IM} = γ_{PL} = 1.75. The values for γ_{DC} and γ_{DW} have a maximum and minimum value.

Therefore, for Strength I Limit State:

$$Q = 1.0\ [\ 1.25(DC) + 1.50(DW) + 1.75((LL + IM) + PL)\ ]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads. The load factors shown for DC and DW are maximum values. Use maximum or minimum values as shown in **LRFD [Table 3.4.1-2]** to calculate the critical force effect.

18.3.3.2 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for Strength Limit State **LRFD [5.5.4.2]** are:

- ϕ = 0.90 for flexure & tension (for tension-controlled reinforced concrete sections as defined in **LRFD [5.6.2.1]**)
- ϕ = 0.90 for shear and torsion

The factored resistance, R_r (M_r, V_r, T_{cap}), associated with the list of items to be designed/checked using Strength I Limit State in 18.3.3, are described in the following sections.

18.3.3.2.1 Moment Capacity

Stress is assumed proportional to strain below the proportional limit on the stress-strain diagram. Tests have shown that at high levels of stress in concrete, stress is not proportional to strain. Recognizing this fact, strength analysis takes into account the nonlinearity of the stress-strain diagram. This is accomplished by using a rectangular stress block to relate the concrete compressive stress distribution to the concrete strain. The compressive stress block has a uniform value of $\alpha_1 \cdot f'_C$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance a = $\beta_1 \cdot (c)$ from the extreme compression fiber. The distance (c) is measured perpendicular to the neutral axis. The factor α_1 shall be taken as 0.85 for concrete strengths not exceeding 10.0 ksi and the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi **LRFD [5.6.2.2]**. Strength predictions using this method are in agreement with strength test results. The representation of these assumptions is shown in Figure 18.3-1.

The moment capacity (factored resistance) of concrete components shall be based on the conditions of equilibrium and strain compatibility, resistance factors as specified in **LRFD [5.5.4.2]** and the assumptions outlined in **LRFD [5.6.2]**.

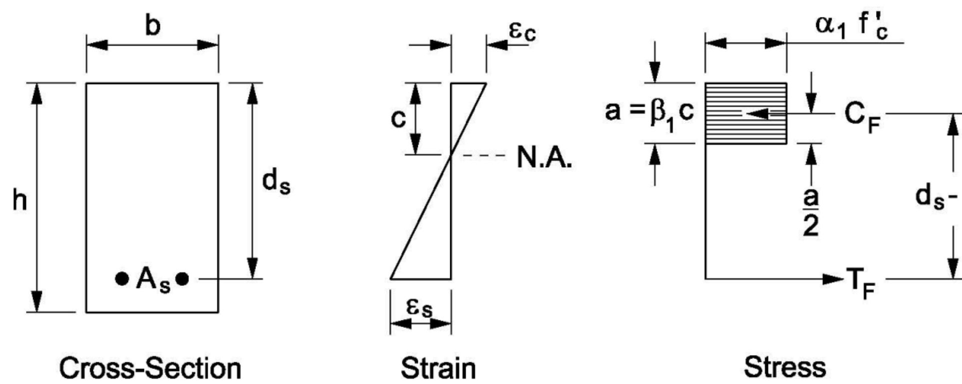


Figure 18.3-1
Stress / Strain on Cross - Section

Referring to Figure 18.3-1, the internal force equations are:

$C_F = \alpha_1 \cdot (f'_c)\ (b)\ (a) = 0.85\ (f'_c)\ (b)\ (a)$

$T_F = (A_s)\ (f_s)$

By equating C_F to T_F, and solving for the compressive stress block depth, (a), gives:

$a = A_s\ f_s\ /\ 0.85\ (f'_c)\ (b)$

Use ($f_s = f_y$) when the steel yields prior to crushing of the concrete. To check for yielding, assume ($f_s = f_y$) and calculate the value for (a). Then calculate the value for $c = a / \beta_1$ and d_s as shown in Figure 18.3-1. If c / d_s does not exceed the value calculated below, then the reinforcement has yielded and the assumption is correct, as stated in **LRFD [5.6.2.1]**.

$c / d_s \leq 0.003 / (0.003 + \varepsilon_{cl})$

ε_{cl} = compression controlled strain limit

for f_y = 60 ksi, ε_{cl} is 0.0020 per **LRFD [Table C5.6.2.1-1]**

if $c / d_s \leq 0.6$, then the reinforcement (f_y = 60 ksi) will yield and ($f_s = f_y$)

For rectangular sections, the nominal moment resistance, M_n , (tension reinforcement only) equals: **LRFD [5.6.3.2.3]**

$M_n = A_s\ f_s\ (d_s - a/2)$

The factored resistance, M_r , or moment capacity, shall be taken as: **LRFD [5.6.3.2.1]**

$M_r = \phi\ M_n = \phi\ A_s\ f_s\ (d_s - a/2)$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$$M_r = (0.9)\, A_s\, f_s\, (d_s - a/2)$$

18.3.3.2.2 Shear Capacity

The nominal shear resistance, V_n , for two-way action, shall be determined as: **LRFD [5.7.1.4, 5.12.8.6.3]**

$$V_n = (0.063 + 0.126 / \beta_c)\, \lambda\, (f'_c)^{1/2}\, b_o\, d_v \le 0.126\, \lambda\, (f'_c)^{1/2}\, b_o\, d_v \quad \text{(kips)}$$

Where:

f'_c	=	4.0 ksi (for concrete slab bridges)
β_c	=	ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted
d_v	=	effective shear depth as determined in **LRFD [5.7.2.8]** (in)
b_o	=	perimeter of the critical section (in)
λ	=	conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

The factored resistance, V_r , or shear capacity, shall be taken as: **LRFD [5.7.2.1]**

$$V_r = \phi\, V_n$$

The resistance factor, ϕ, is 0.90, therefore:

$$V_r = (0.9)\, V_n$$

18.3.3.2.3 Uplift Check

The check of uplift at abutments does not use a factored resistance, but compares factored dead load and live load reactions.

18.3.3.2.4 Tensile Capacity – Longitudinal Reinforcement

The nominal tensile resistance, T_{nom} , for an area, A_s , of developed reinforcement, equals:

$$T_{nom} = A_s\, f_y$$

The factored resistance, T_{cap} , or tensile capacity, shall be taken as:

$$T_{cap} = \phi\, T_{nom} = \phi\, A_s\, f_y$$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$T_{cap} = (0.9)\ A_s\ f_y$

18.3.4 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Service I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for crack control criteria
- Checking transverse slab reinforcement over the piers for crack control criteria
- Checking live load deflection criteria
- Checking dead load deflection (camber) criteria

18.3.4.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Service I Limit State will be used to analyze the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and/or appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, where applicable, defined in 18.4.3.2.

For Service I Limit State, the values of γ_i for each applied load, are found in **LRFD [Table 3.4.1-1]** and their values are: $\gamma_{DC} = \gamma_{DW} = \gamma_{LL+IM} = \gamma_{PL} = 1.0$

Therefore, for Service I Limit State:

$$Q = 1.0\ [\ 1.0(DC) + 1.0(DW) + 1.0((LL + IM) + PL)\ ]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads.

18.3.4.2 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

The factored resistance, R_r , associated with the list of items to be checked using Service I Limit State in 18.3.4, are described in the following sections.

18.3.4.2.1 Crack Control Criteria

All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

18.3.4.2.2 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits. The Bureau of Structures limits live load deflections for concrete slab structures to L/1200. The deflections are based on entire slab width acting as a unit and gross moment of inertia, I_g .

The nominal resistance, R_n , or deflection limit, is:

$$R_n = L/1200$$

Where:

L = span length

The factored resistance, R_r , is:

$$R_r = \phi\ R_n = \phi\ (L/1200)$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0)\ R_n = (L/1200)$$

18.3.4.2.3 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . Bureau of Structures calculates full camber based on multiplying the dead load deflection values by a factor of three. A maximum allowable camber has been set for simple-span slabs and continuous-span slabs as shown in 18.4.4.2.

The nominal resistance, R_n , or deflection limit, is:

$$R_n = (\text{maximum allowable camber}) / 3$$

The factored resistance, R_r , is:

$$R_r = \phi\ R_n = \phi\ (\text{maximum allowable camber}) / 3$$

The resistance factor, ϕ, is 1.00, therefore:

$R_r = (1.0)\ R_n =$ (maximum allowable camber) / 3

18.3.5 Fatigue Limit State

Fatigue I Limit State shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles **LRFD [1.3.2.3]**. The Fatigue I Limit State is intended to limit crack growth under repetitive loads to prevent fracture of the reinforcement during the design life of the bridge. The factored force effect (stress range), Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

For fatigue considerations, concrete members shall satisfy: **LRFD [5.5.3.1]**

$$\eta_i\ \gamma_i\ (\Delta f) \leq (\Delta F)_{TH}$$

Where:

γ_i	=	Load factor for Fatigue I Limit State
Δf	=	Force effect, live load stress range due to the passage of the fatigue truck (ksi)
$(\Delta F)_{TH}$	=	Constant-amplitude fatigue threshold (ksi)

Fatigue I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for fatigue stress range criteria

18.3.5.1 Factored Loads (Stress Range)

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Fatigue I Limit State will be used to analyze the structure for force effects, $Q_i = (\Delta f)$, due to applied (Fatigue Truck) live load, LL and IM, defined in 18.4.3.1.

For Fatigue I Limit State, the value of γ_i for the applied live load, is found in **LRFD [Table 3.4.1-1]** and its value is $\gamma_{LL+IM} = 1.75$.

Therefore, for Fatigue I Limit State:

$$Q = 1.0\ [\ 1.75(LL + IM)\ ]$$

Where LL and IM represent force effects, Δf, due to these applied loads.

18.3.5.2 Factored Resistance

The resistance factor, ϕ, for Fatigue Limit State, is found in **LRFD [C1.3.2.1]** and its value is 1.00 .

18.3.5.2.1 Fatigue Stress Range

The nominal resistance, $R_n = (\Delta F)_{TH}$, for fatigue stress range (for straight reinforcement), is: **LRFD [5.5.3.2]**

$$R_n = (\Delta F)_{TH} = 26 - 22\, f_{min} / f_y \quad \text{(ksi)}$$

Where:

f_{min}	=	the minimum stress resulting from the factored Fatigue Truck live load, combined with the stress from the dead loads on the structure; positive if tension, negative if compression (ksi)
f_y	=	minimum yield strength (ksi), not to be taken less than 60 ksi nor greater than 100 ksi

The factored resistance, R_r (for f_y = 60 ksi), is:

$$R_r = \phi\, R_n = \phi\, (26 - 0.37\, f_{min})$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0)\, R_n = 26 - 0.37\, f_{min} \quad \text{(ksi)}$$

18.4 Concrete Slab Design Procedure

18.4.1 Trial Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, skew, roadway width, etc.. The selection of the type of concrete slab structure (haunched / flat) is a function of the span lengths selected. Recommended span length ranges and corresponding structure type are shown for single-span and multiple-span slabs in Figure 18.2-1. Optimum span ratios for multiple-span slabs are suggested in 18.2.3. Knowing the span lengths and the structure type, a trial slab depth can be obtained from Table 18.2-1.

For haunched slabs, the haunch depth, D_{haunch} , is proportional to the slab depth, d_{slab} , outside the haunch. A trial haunch depth can be selected as:

$$D_{haunch} = d_{slab} / 0.6$$

An economical haunch length, L_{haunch} , measured from C/L of pier to end of haunch, can be approximated between (0.15 L_2 to 0.18 L_2), where L_2 is the length of an interior span.

NOTE: With preliminary structure sizing complete, check to see if structure exceeds limitations in 18.1.2.

18.4.2 Dead Loads (DC, DW)

Dead loads (permanent loads) are defined in **LRFD [3.3.2]**. Concrete dead load is computed by using a unit weight of 150 pcf, with no adjustment in weight for the bar steel reinforcement.

DC = dead load of structural components and any nonstructural attachments

DW = dead load of future wearing surface (F.W.S.) and utilities

The slab dead load, DC_{slab} , and the section properties of the slab, do not include the ½ inch wearing surface. A post dead load, DW_{FWS} , of 20 psf, for possible future wearing surface (F.W.S.), is required in the design by the Bureau of Structures. The ½ inch wearing surface load, $DC_{1/2\text{" }WS}$, of 6 psf must also be included in the design of the slab.

Dead loads, DC, from parapets, medians and sidewalks are uniformly distributed across the full width of the slab when designing an interior strip. For the design of exterior strips (edge beams), any of these dead loads, DC, that are located directly over the exterior strip width and on the cantilevered portion of sidewalks, shall be applied to the exterior strip. For both interior and exterior strips, the future wearing surface, DW, located directly over the strip width shall be applied to it. See 17.2.7 for the distribution of dead loads.

18.4.3 Live Loads

18.4.3.1 Vehicular Live Load (LL) and Dynamic Load Allowance (IM)

The *AASHTO LRFD* Specifications contain several live load components (see 17.2.4.2) that are combined and scaled to create live load combinations that apply to different Limit States **LRFD [3.6.1]**.

The live load combinations used for design are:

LL#1:	Design Tandem (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#2:	Design Truck (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#3:	90% [Double Design Trucks (+ IM) + Design Lane Load]	**LRFD [3.6.1.3.1]**
LL#4:	Fatigue Truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5:	Design Truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6:	25% [Design Truck (+ IM)] + Design Lane Load	**LRFD [3.6.1.3.2]**

Table 18.4-1
Live Load Combinations

The dynamic load allowance, IM, **LRFD [3.6.2]** for the live load combinations above, is shown in Table 18.4-2.

Where (IM) is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load. (IM) is not applied to the Design Lane Load.

The live load combinations are applied to the Limit States as shown in Table 18.4-2.

The live load force effect, Q_i , shall be taken as the largest from the live loads shown in Table 18.4-2 for that Limit State.

Strength I Limit State: [1]	LL#1 , LL#2 , LL#3 [2]	IM = 33%
Service I Limit State: [1] (for crack control criteria)	LL#1 , LL#2 , LL#3 [2]	IM = 33%
Service I Limit State: (for LL deflection criteria)	LL#5 , LL#6	IM = 33%
Fatigue I Limit State: [3]	LL#4 (single Fatigue Truck)	IM = 15%

Table 18.4-2
Live Loads for Limit States

[1] Load combinations shown are used for design of interior strips and exterior strips without raised sidewalks, as shown in Figures 17.2-6 to 10. For an exterior strip with a raised sidewalk,

use Design Lane Load portion of LL#2 for Live Load Case 1 and use Design Truck (+IM) portion of LL#2 for Live Load Case 2, as shown in Figure 17.2-11.

[2] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

[3] Used for design of interior strip only.

18.4.3.2 Pedestrian Live Load (PL)

For bridges designed for both vehicular and pedestrian live load, a pedestrian live load, PL, of 75 psf is used. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, see *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for live load. The dynamic load allowance, IM, is not applied to pedestrian live loads **LRFD [3.6.2]**.

Pedestrian loads are not applied to an interior strip for its design. For the design of exterior strips (edge beams), any pedestrian loads that are located directly over the exterior strip width and on the cantilevered portion of the sidewalk, shall be applied to the exterior strip. See 17.2.7 for the distribution of pedestrian live loads.

18.4.4 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

18.4.4.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**. Live load deflections for concrete slab structures are limited to L/1200, by the Bureau of Structures. The live load deflection, Δ_{LL+IM} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.3.1 for Service I Limit State.

Place live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**. Use gross moment of inertia, I_g , based on entire slab width acting as a unit. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2. The factored resistance, R_r , is described in 18.3.4.2.2.

Then check that, $\Delta_{LL+IM} \leq R_r$ is satisfied.

18.4.4.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.6.3.5.2]**. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . All dead loads are to be uniformly distributed across the width of the slab. These deflections are increased to provide for the time-dependent deformations of creep and shrinkage. Bureau of Structures currently calculates full camber as three times the dead load deflection. Most of the excess camber is dissipated during the first year of service, which is the time period that the majority of creep and shrinkage deflection occurs. Noticeable excess deflection or structure sag can normally be attributed to falsework settlement. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2 . The dead load deflection, Δ_{DL} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.2. The factored resistance, R_r , is described in 18.3.4.2.3.

WisDOT exception to AASHTO:

Calculating full camber as three times the dead load deflection, as stated in paragraph above, is an exception to **LRFD [5.6.3.5.2]**. This exception, used by the Bureau of Structures, is based on field observations using this method.

Then check that, $\Delta_{DL} \leq R_r$ is satisfied.

A "Camber Diagram" is shown in the plans on the "Superstructure" sheet. Provide camber values, as well as centerline and edge of slab elevations, at 0.1 points of all spans.

Simple-Span Concrete Slabs:

Maximum allowable camber for simple-span slabs is limited to 2 ½ inches. For simple-span slabs, Bureau of Structures practice indicates that using a minimum slab depth (ft) from the equation 1.1(S + 10) / 30, (where S is span length in feet), and meeting the live load deflection and dead load deflection (camber) limits stated in this section, provides an adequate slab section for most cases.

WisDOT exception to AASHTO:

The equation for calculating minimum slab depth for simple-spans, as stated in paragraph above, is an exception to **LRFD [Table 2.5.2.6.3-1]**. This exception, used by the Bureau of Structures, is based on past performance using this equation.

Continuous-Span Concrete Slabs:

Maximum allowable camber for continuous-span slabs is 1 ¾ inches.

18.4.5 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

18.4.5.1 Interior Strip

Equivalent interior strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]**.

The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load.

Single-Lane Loading: $E = 10.0 + 5.0\,(L_1\,W_1)^{1/2}$

Multi-Lane Loading: $E = 84.0 + 1.44(L_1\,W_1)^{1/2} \leq 12.0(W)/N_L$

Where:

E = equivalent distribution width (in)

L_1 = modified span length taken equal to the lesser of the actual span or 60.0 ft (ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft)

W = physical edge to edge width of bridge (ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

18.4.5.1.1 Strength and Service Limit State

Use the smaller equivalent width (single-lane or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E}$$

Where:

E = equivalent distribution width (ft)

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore aren't used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.5.1.2 Fatigue Limit State

Use equivalent widths from single-lane loading to check fatigue stress range criteria. For the Fatigue Limit State only one design truck (Fatigue Truck) is present **LRFD [3.6.1.4]**. Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor, m, which are present in the equation for equivalent width, E, **LRFD [3.6.1.1.2]**.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E(1.20)}$$

Where:

E = equivalent distribution width (ft)

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.5.2 Exterior Strip

Equivalent exterior strip widths for slab bridges are covered in **LRFD [4.6.2.1.4]**.

For Exterior Strips without Raised Sidewalks:

The exterior strip width, E, is assumed to carry one wheel line and a tributary portion of design lane load (located directly over the strip width) as shown in Figures 17.2-7 and 17.2-9.

E equals the distance between the edge of the slab and the inside face of the barrier, plus 12 inches, plus ¼ of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width, E, shall not exceed either ½ the full strip width or 72 inches.

Use the smaller equivalent width (single-lane or multi-lane), for full strip width, when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The multiple presence factor, m, has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor **LRFD [3.6.1.1.2]**.

For Exterior Strips with Raised Sidewalks:

The exterior strip width, E, is to carry a tributary portion of design lane load (when its located directly over the strip width) as in Live Load Case 1 or one wheel line as in Live Load Case 2, as shown in Figure 17.2-11.

The exterior strip width, E, shall be 72 inches.

18.4.5.2.1 Strength and Service Limit State

The distribution factor, DF, is computed for a design slab width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to axle loads:

$$DF = \frac{(1 \text{ wheel line})}{(2 \text{ wheel lines/lane})(E)}$$

Where:

E = equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(SWL)}{(10\,\text{ft lane load width})}\right]}{(E)}$$

Where:

E = equivalent distribution width (ft)

SWL = Slab Width Loaded (with lane load) (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of barrier) or

E – (distance from edge of slab to inside face of raised sidewalk)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.10.8.2.3, 5.10.8.4.2a]**.

18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.6.2]**.

The area of longitudinal slab reinforcement, A_s , should be designed for strength at maximum moment locations along the structure, and for haunched slab structures, checked for strength at the haunch/slab intercepts. The area should also be checked for strength at bar reinforcement cutoff locations. This reinforcement should be designed for interior and exterior strips (edge beams) in both positive and negative moment regions. The reinforcement in the exterior strip is always equal to or greater than that required for the slab in an interior strip. Compare the reinforcement to be used for each exterior strip and select the strip with the largest amount of reinforcement (in^2/ft). Use this reinforcement pattern for both exterior strips to keep the bar layout symmetrical. Concrete parapets, curbs, sidewalks and other appurtenances are not to be considered to provide strength to the edge beam **LRFD [9.5.1]**. The total factored moment, M_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Then calculate the coefficient of resistance, R_u :

$$R_u = M_u / \phi\, b\, d_s^2$$

Where:

ϕ	=	0.90 (see 18.3.3.2)
b	=	12 in (for a 1 foot design slab width)
d_s	=	slab depth (excl. ½ inch wearing surface) – bar clearance – ½ bar diameter (in)

Calculate the reinforcement ratio, ρ, using (R_u vs. ρ) Table 18.4-3 .

Then calculate required area,

$$A_s = \rho\, (b)\, (d_s)$$

Area of bar reinforcement per foot of slab width can be found in Table 18.4-4 .

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_u \le M_r$ is satisfied.

The area of longitudinal reinforcement, A_s , should also be checked for moment capacity (factored resistance) along the structure, to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1. See Chapter 45 for details on checking the capacity of the structure for this Permit Vehicle.

18.4.6.2 Check for Fatigue

Fatigue Limit State considerations and assumptions are detailed in **LRFD [5.5.3, 5.6.1, 9.5.3]**

The area of longitudinal slab reinforcement, A_s , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than 1.75 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.75 times the fatigue load is tensile and exceeds $0.095\ (f'_c)^{1/2}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.6.1, 5.6.7]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$$s \leq (700\ \gamma_e / \beta_s f_{ss}) - 2\ (d_c) \quad \text{(in)}$$

LRFD [5.6.7]

in which:

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

Where:

γ_e = 1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e = 0.75 for Class 2 exposure condition (top reinforcement)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, (in). For top reinforcement, d_c , should not include the ½" wearing surface

f_{ss} = tensile stress in steel reinforcement (ksi) $\leq 0.6 f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h = overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.6.3.3]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) 1.33 M_u

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

f_r = $0.24 \lambda (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

I_g = gross moment of Inertia (in^4)

c = effective slab thickness/2 (in)

M_u = total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Select lowest value of [M_{cr} (or) 1.33 M_u] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_L \leq M_r$ is satisfied.

18.4.6.5 Bar Cutoffs

One-half of the bar steel reinforcement required for maximum moment can be cut off at a point, where the remaining one-half has the moment capacity, or factored resistance, M_r , equal to the total factored moment, M_u , at that point. This is called the theoretical cutoff point.

Select tentative cutoff point at theoretical cutoff point or at a distance equal to the development length from the point of maximum moment, whichever is greater. The reinforcement is extended beyond this tentative point for a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. This cutoff point is acceptable, if it satisfies fatigue and crack control criteria. The continuing bars must be fully developed at this point **LRFD [5.10.8.1.2a]**.

18.4.6.5.1 Positive Moment Reinforcement

At least one-third of the maximum positive moment reinforcement in simple-spans and one-fourth of the maximum positive moment reinforcement in continuous-spans is extended along the same face of the slab beyond the centerline of the support **LRFD [5.10.8.1.2b]**.

18.4.6.5.2 Negative Moment Reinforcement

For negative moment reinforcement, the second tentative cutoff point is at the point of inflection. At least one-third of the maximum negative moment reinforcement must extend beyond this point for a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater **LRFD [5.10.8.1.2c]**.

18.4.7 Transverse Slab Reinforcement

18.4.7.1 Distribution Reinforcement

Distribution reinforcement is placed transversely in the bottom of the slab, to provide for lateral distribution of concentrated loads **LRFD [5.12.2.1]**. The criteria for main reinforcement parallel to traffic is applied. The amount of distribution reinforcement is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \leq 50\% \text{ maximum}$$

Where:

L = span length (ft)

The above formula is conservative when applied to slab structures. This specification was primarily drafted for the relatively thin slabs on stringers.

18.4.7.2 Reinforcement in Slab over Piers

If the concrete superstructure rests on a pier cap (with columns) or directly on columns, design of transverse slab reinforcement over the pier is required. A portion of the slab over the pier is designed as a continuous transverse slab member (beam) along the centerline of the substructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a concrete slab, the width of the transverse slab member is equal to one-half the center to center spacing between columns (or 8 foot maximum) for the positive moment zone. The width equals the diameter of the column plus 6 inches for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Reference is made to the design example in 18.5 of this chapter for computations relating to transverse reinforcement in slab over the piers.

18.4.8 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.

The area, A_s , of reinforcement per foot for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.6]**

$$A_s \geq 1.30\ (b)\ (h)\ /\ 2\ (b+h)\ (f_y) \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	area of reinforcement in each direction and on each face (in^2/ft)
b	=	least width of component section (in)
h	=	least thickness of component section (in)
f_y	=	specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches. For components greater than 36 inches thick, the spacing shall not exceed 12 inches.

All longitudinal reinforcement and transverse reinforcement in the slab must exceed required A_s (on each face and in each direction), and not exceed maximum spacing.

18.4.9 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.12.2.1]**.

18.4.10 Longitudinal Reinforcement Tension Check

The tensile capacity check of longitudinal reinforcement on the flexural tension side of a member is detailed in **LRFD [5.7.3.5]**.

The area of longitudinal reinforcement (in bottom of slab), A_s , should be checked for tensile capacity at the abutments, for dead load and (HL-93) live load on interior and exterior strips. The reinforcement at these locations shall have the capacity to resist the tension in the reinforcement produced by shear.

The factored shear, V_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. The factored tension force, T_{fact} , from shear, to be resisted is from **LRFD [Eq'n. 5.7.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = [\ V_u / \phi_v\ ] \cot \theta$$

Assume a diagonal crack would start at the inside edge of the bearing area. Assume the crack angle, θ, is 35 degrees. Calculate the distance from the bottom of slab to center of tensile reinforcement. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement. Find the development length, ℓ_d , from Table 9.9-2, Chapter 9.

The nominal tensile resistance, T_{nom} , of the longitudinal bars at the crack location is:

$$T_{nom} = A_s f_y [\ D_{crack} - (\text{end cover})] / \ell_d \ \leq \ A_s f_y$$

Then check that, $T_{fact} \leq T_{nom}$ is satisfied.

If the values for T_{fact} and T_{nom} are close, the procedure for determining the crack angle, θ, as outlined in **LRFD [5.7.3.4.2]** should be used.

18.4.11 Uplift Check

Check for uplift at the abutments for (HL-93) live loads **LRFD [C3.4.1, 5.5.4.3]**. Compare the factored dead load reaction to the factored live load reaction. The reactions shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Place (HL-93) live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**.

18.4.12 Deflection Joints and Construction Joints

The designer should locate deflection joints in sidewalks and parapets on concrete slab structures according to the Standard *Vertical Face Parapet 'A'* in Chapter 30.

Refer to Standards *Continuous Haunched Slab* and *Continuous Flat Slab* in Chapter 18, for recommended construction joint guidelines.

18.4.13 Reinforcement Tables

Table 18.4-3 applies to: Rectangular Sections with Tension Reinforcement only

- Reinforcement Yield Strength (f_y) = 60,000 psi
- Concrete Compressive Strength (f'_c) = 4,000 psi

R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ
117.9	0.0020	335.6	0.0059	537.1	0.0098	722.6	0.0137	892.0	0.0176
123.7	0.0021	340.9	0.0060	542.1	0.0099	727.2	0.0138	896.1	0.0177
129.4	0.0022	346.3	0.0061	547.1	0.0100	731.7	0.0139	900.2	0.0178
135.2	0.0023	351.6	0.0062	552.0	0.0101	736.2	0.0140	904.4	0.0179
141.0	0.0024	357.0	0.0063	556.9	0.0102	740.7	0.0141	908.5	0.0180
146.7	0.0025	362.3	0.0064	561.8	0.0103	745.2	0.0142	912.5	0.0181
152.4	0.0026	367.6	0.0065	566.7	0.0104	749.7	0.0143	916.6	0.0182
158.1	0.0027	372.9	0.0066	571.6	0.0105	754.2	0.0144	920.7	0.0183
163.8	0.0028	378.2	0.0067	576.5	0.0106	758.7	0.0145	924.8	0.0184
169.5	0.0029	383.5	0.0068	581.4	0.0107	763.1	0.0146	928.8	0.0185
175.2	0.0030	388.8	0.0069	586.2	0.0108	767.6	0.0147	932.8	0.0186
180.9	0.0031	394.1	0.0070	591.1	0.0109	772.0	0.0148	936.9	0.0187
186.6	0.0032	399.3	0.0071	595.9	0.0110	776.5	0.0149	940.9	0.0188
192.2	0.0033	404.6	0.0072	600.8	0.0111	780.9	0.0150	944.9	0.0189
197.9	0.0034	409.8	0.0073	605.6	0.0112	785.3	0.0151	948.9	0.0190
203.5	0.0035	415.0	0.0074	610.4	0.0113	789.7	0.0152	952.9	0.0191
209.1	0.0036	420.2	0.0075	615.2	0.0114	794.1	0.0153	956.8	0.0192
214.8	0.0037	425.4	0.0076	620.0	0.0115	798.4	0.0154	960.8	0.0193
220.4	0.0038	430.6	0.0077	624.8	0.0116	802.8	0.0155	964.7	0.0194
225.9	0.0039	435.8	0.0078	629.5	0.0117	807.2	0.0156	968.7	0.0195
231.5	0.0040	441.0	0.0079	634.3	0.0118	811.5	0.0157	972.6	0.0196
237.1	0.0041	446.1	0.0080	639.0	0.0119	815.8	0.0158	976.5	0.0197
242.7	0.0042	451.3	0.0081	643.8	0.0120	820.1	0.0159	980.4	0.0198
248.2	0.0043	456.4	0.0082	648.5	0.0121	824.5	0.0160	984.3	0.0199
253.7	0.0044	461.5	0.0083	653.2	0.0122	828.8	0.0161	988.2	0.0200
259.3	0.0045	466.6	0.0084	657.9	0.0123	833.1	0.0162	992.1	0.0201
264.8	0.0046	471.7	0.0085	662.6	0.0124	837.3	0.0163	996.0	0.0202
270.3	0.0047	476.8	0.0086	667.3	0.0125	841.6	0.0164	999.8	0.0203
275.8	0.0048	481.9	0.0087	671.9	0.0126	845.9	0.0165	1003.7	0.0204
281.3	0.0049	487.0	0.0088	676.6	0.0127	850.1	0.0166	1007.5	0.0205
286.8	0.0050	492.1	0.0089	681.3	0.0128	854.3	0.0167	1011.3	0.0206
292.2	0.0051	497.1	0.0090	685.9	0.0129	858.6	0.0168	1015.1	0.0207
297.7	0.0052	502.2	0.0091	690.5	0.0130	862.8	0.0169	1018.9	0.0208
303.1	0.0053	507.2	0.0092	695.1	0.0131	867.0	0.0170	1022.7	0.0209
308.6	0.0054	512.2	0.0093	699.7	0.0132	871.2	0.0171	1026.5	0.0210
314.0	0.0055	517.2	0.0094	704.3	0.0133	875.4	0.0172	1030.3	0.0211
319.4	0.0056	522.2	0.0095	708.9	0.0134	879.5	0.0173	1034.0	0.0212
324.8	0.0057	527.2	0.0096	713.5	0.0135	883.7	0.0174	1037.8	0.0213
330.2	0.0058	532.2	0.0097	718.1	0.0136	887.9	0.0175	----	----

Table 18.4-3

R_u (psi) vs. ρ

R_u = coefficient of resistance (psi) = $M_u / \phi\, b\, d_s^2$ ρ = reinforcement ratio = $A_s / b\, d_s$

Table 18.4-4 can be used to select bar size and bar spacing to provide an adequate area of reinforcement to meet design requirements.

Bar Size Number	Nominal Dia. Inches	4 1/2"	5"	5 1/2"	6"	6 1/2"	7"	7 1/2"	8"	8 1/2"	9"	10"	12"
4	0.500	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.20
5	0.625	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.31
6	0.750	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.44
7	0.875	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.60
8	1.000	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.79
9	1.128	--	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.00
10	1.270	--	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.27
11	1.410	--	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.56

Table 18.4-4

Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

18.5 Design Example