PART 2

THE AASHTO LRFD SPECIFICATIONS

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1.0 INTRODUCTION

The AASHTO LRFD Specifications are written based on probabilistic limit state theory with several load combinations listed. These load combinations correspond to four limit states, Service, Fatigue, Fracture, Strength and Extreme-Event.

Service limit states are restrictions on stress, deformation and crack width under regular service conditions. They are intended to allow the bridge to perform acceptably for its service life.

Fatigue and fracture limit states are restrictions on stress range under regular service conditions reflecting the number of expected stress range excursions. They are intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

Strength limit states are intended to ensure that strength and stability, both local and global, are provided to resist the statistically significant load combinations that a bridge will experience in its design life. Extensive distress and structural damage may occur under strength limit states, but overall structural integrity is expected to be maintained.

Extreme event limit states are intended to ensure the structural survival of a bridge during a major earthquake, or when collided by a vessel, vehicle or ice flow, or where the foundation is subject to the scour which would accompany a flood of extreme recurrence, usually considered to be 500 years. They are considered to be unique occurrences whose return period is significantly greater than the design life of the bridge.

1.1 Limit State

Definition: A condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed.

Requirement — $\eta_i \gamma_i Q_i \le \phi R_n = R_r$ (LRFD Eq. 1.3.2.1-1)

(a) For loads for which a <u>maximum</u> value of γ_i is appropriate:

$\eta_i = \eta_D \eta_R \eta_I \ge 0.95$ (LRFD Eq. 1.3.2.1-2)

(b) For loads for which a <u>minimum</u> value of γ_i is appropriate:

$\eta_i = 1/\eta_D \eta_R \eta_I \le 1.0$ (LRFD Eq. 1.3.2.1-3)

(1) η_D = Ductility factor

$\ge$ 1.05 Strength Limit State; non-ductile components and connections

= 1.00 Strength Limit State; conventional designs and details complying with these specifications And all other Limit States

$\ge$ 0.95 Strength Limit State; additional ductility - enhancing measures

(2) η_R = Redundancy factor

$\ge$ 1.05 Strength Limit State; non-redundant members

= 1.00 Strength Limit State; conventional levels of redundancy And all other Limit States

$\ge$ 0.95 Strength Limit State; exceptional levels of redundancy

(3) η_I = Operational Importance

$\geq$ 1.05 Strength Limit State; important bridges ("critical" or "essential" bridges with earthquakes 475-year and 2500-year return periods, respectively)

= 1.00 Strength Limit State; typical bridges
And all other Limit States

$\geq$ 0.95 Strength Limit State; relatively less important bridges

Example: Major Bridge. Multi-girder Steel. (redundant member)

Fatigue	η	= (1.0)(1.0)(1.0)	= 1.0
Strength	η	= (1.0)(1.0)(1.05)	= 1.05
Others	η	= (1.0)(1.0)(1.0)	= 1.00

Minor Bridge.

Fatigue	η	= (1.0)(1.0)(1.0)	= 1.00
Strength	η	= (1.0)(1.0)(0.95)	= 0.95
Others	η	= (1.0)(1.0)(1.0)	= 1.0

1.2. Load Combinations

The permanent and transient loads and forces listed in Section 1.6 shall be considered in the various load combinations. The complete list is in Tables 1-1 and 1-2 (LRFD Table 3.4.1-1 & Table 3.4.1-2).

$$Q = \sum \eta_i \gamma_i q_i \qquad \text{(LRFD Eq. 3.4.1-1)}$$

(1) Strength I - Normal vehicle, no wind

$\gamma_P D + 1.75\,\mathrm{L} + ...$ $\qquad$ γ_P - DC = 1.25 - 0.9
DW = 1.5 - 0.85

(2) Strength II - Permit vehicle, no wind

$\gamma_P D + 1.35\,\mathrm{L} + ...$

(3) Strength III - No live load, max. wind

$\gamma_P D + 1.4\mathrm{WS} + ...$

(4) Strength IV - Dead load only (for large bridge)

γ_P D + ... γ_P - DC = 1.5 - 0.9

(5) Strength V - Normal vehicle with 55mph Wind

$\gamma_P D$ + 1.35 L + 0.4 WS + 1.0 WL + ...

(6) Extreme I - Earthquake

$\gamma_P D + \gamma_{EQ} L + EQ$

(7) Extreme II - Ice load, collision and certain hydraulic events

$\gamma_P D$ + 0.5 L + Max. (IC, CT, CV) + ...

(8) Service I - Normal operation with 55mph Wind

D + L + 0.3 WS + 1.0WL + ...

(9) Service II - Overload event, intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

D + 1.3 L + ...

(10) Service III - Tension in prestressed concrete superstructure

D + 0.8L + . . .

(11) Service IV - Tension is prestressed concrete substructure

D + 0.7 WS + …

(12) Fatigue - Fatigue event, stress range of a single design truck

0.75 L

So:

Superstructure - Check Strength I, II, Service I, II, (III,) Fatigue.

For large bridges, also check Strength III, IV, V, Extreme

Substructure - Check Strength I, II, III, IV, V, Extreme, Service I, II (,III). (Add WA + FR to all the conditions.)

TABLE 1-1 AASHTO LOAD COMBINATION TABLE &

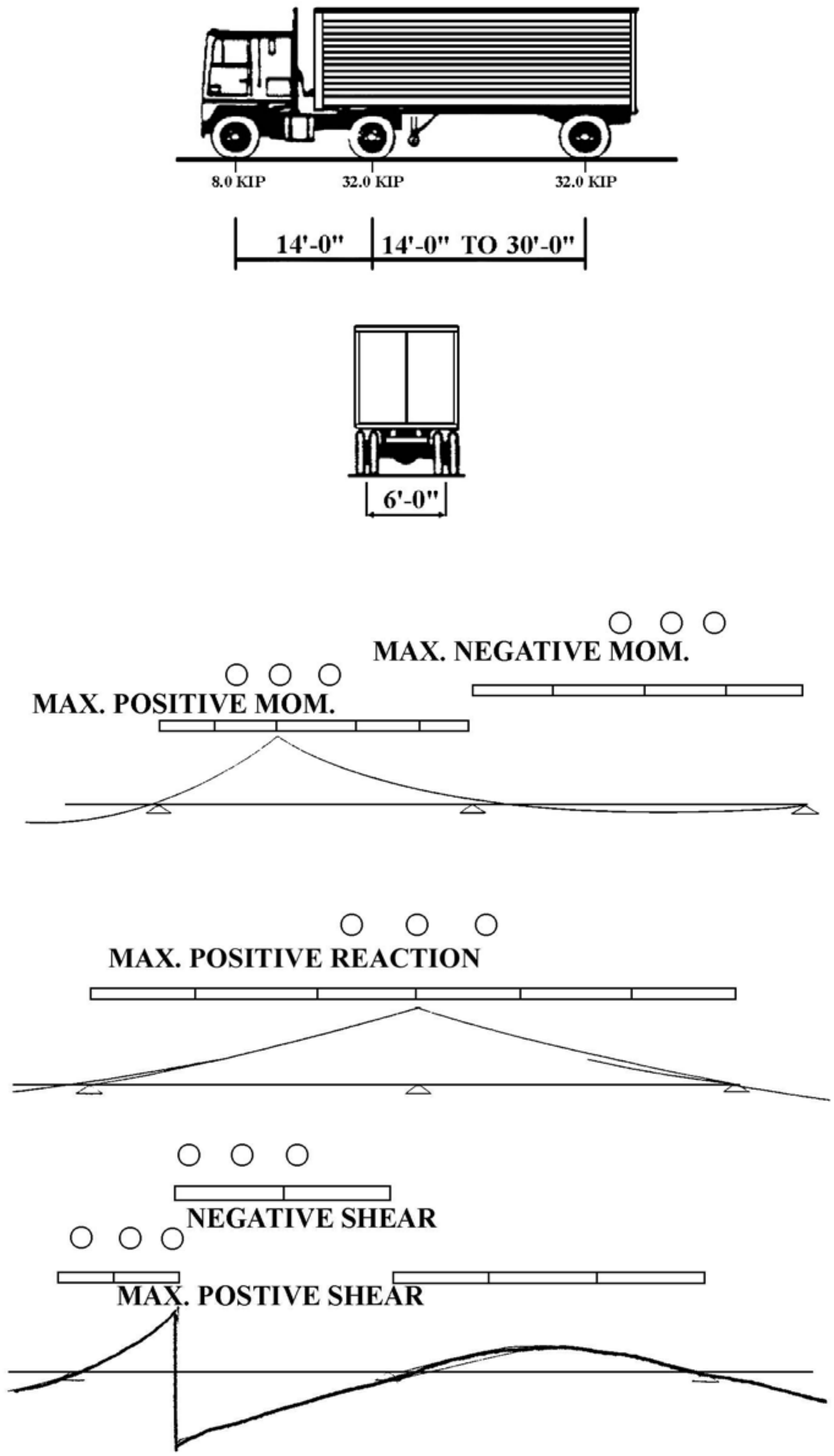
TABLE 1-2 AASHTO LOAD FACTORS FOR PERMANENT LOADS TABLE

Load Combination Limit State	DC/DD DW/EH EV/ES EL	LL/IM CE/BR PL/LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_P	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_P	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_P	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV	γ_P	-	1.00	-	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-V	γ_P	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	γ_P	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	γ_P	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-IV	1.00	-	1.00	0.70	-	1.00	1.00/1.20	-	1.0	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

(AASHTO LRFD TABLE 3.4.1-2 – Load Factors for Permanent Loads, γ_P)

Type of Load		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.5	0.9
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, β Tomlinson Method	1.05	0.30
	Drilled Shafts, O'Neill and Reese(1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• AEP for anchored walls		1.35	N/A
EL: Locked-in Erection Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Structure(Walls and Abutments)		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts		1.50	0.90
ES: Earth Surcharge		1.50	0.75

FIGURE 1-1 LRFD DESIGN VEHICULAR LIVE LOAD, HL-93



1.3. Design Vehicle Live Load (LRFD Art. 3.6.1.2)

The vehicular live loading for LRFD is designated HL-93 (Figure 2-1), which consists of a combination of the:

(1) Design Truck + Design Lane

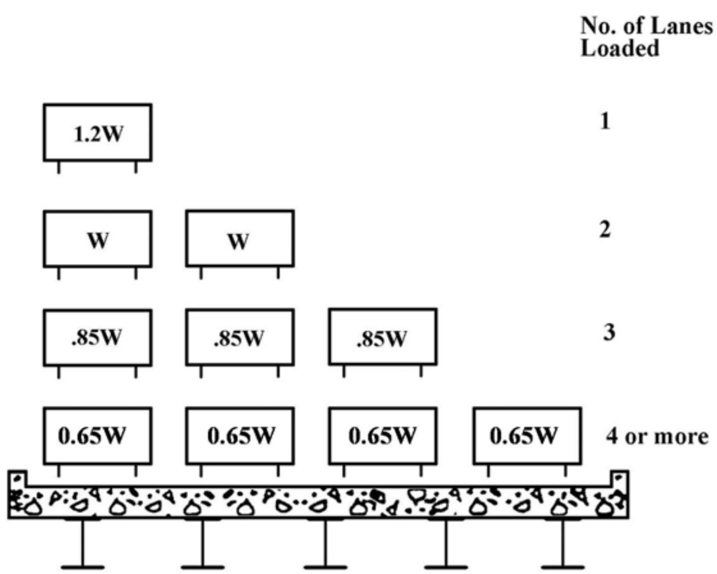
(2) Design Tandem + Design Lane

! For M^- or Reactions at interior piers, two 90% Design Trucks spaced at least 50ft + 90% Design Lane

Multiple presence factors are not to be applied to the fatigue limit state for which one design truck is used, regardless of the number of design lanes. Thus, the factor 1.20 must be removed from the single lane distribution factors when they are used to investigate fatigue.

Number of Loaded Lanes	Multiple Presence Factors, m
1	1.20
2	1.00
3	0.85
>3	0.65

FIGURE 1-2 AASHTO LRFD MULTIPLE PRESENCE FACTORS



1.4. Fatigue Load

Design truck only with constant 30' between 32-kip axles. (LRFD Art. 3.6.1.4)

$ADTT_{single\text{-}lane}$ = p x ADTT

where p = 1.0 for one-lane bridge

= 0.85 for two-lane bridge

= 0.80 for three-lane or more bridge

Class of Highway	ADTT/ADT
Rural Interstate	0.2
Urban Interstate	0.15
Other Rural	0.15
Other Urban	0.10

Example: Rural Interstate 4-lane bridge (Major)

$ADTT_{single\text{-}lane}$ = (0.8)(0.2)ADT = 0.16 ADT

Rural 2-lane bridge (Minor)

$ADTT_{single\text{-}lane}$ = (0.85)(0.15)ADT = 0.1275 ADT

(Max ADT = 20,000 vehicles/lane/day)

1.5. Impact (Dynamic Load Allowance = IM) (LRFD Art. 3.6.2)

Deck Joints — All Limit States IM = 75%

All other components, Fatigue and Fracture Limit State IM = 15%

All other components, All other Limit States IM = 33%

Applied to design truck or tandem only; not to be applied to pedestrian loads or to the design lane load.

1.6. Wind (LRFD Art. 3.8)

(1) On structure: WS

		Windward	Leeward	
(a)	Trusses, Columns and Arches	0.05 ksf (0.30 klf min.)	0.025 ksf (0.15 klf min.)	* H V_D^2 / 10,000
(b)	Beams	0.05 ksf (0.30 klf min.)	NA	
(c)	Large Flat Surfaces	0.04 ksf	NA	

A multi-girder bridge. 55 mph design Wind. d = 5′

$$p = 0.05 \times 5 \times 55^2 / 10{,}000 = 0.075625 \text{ klf}$$

$$= 0.30 \times 55^2 / 10{,}000 = 0.09075 \text{ klf} \quad \leftarrow \text{govern}$$

(2) On Vehicles = WL (100 lb/ft acting 6 ft above the roadway, based on 55 mph)

1.7. Distribution Factor (LRFD Art. 4.6.2.2)

(Steel I-Beams, Prestress Concrete, Concrete T-Beam on Concrete deck)

Definition: The AASHTO Specs permit a simplified method by modeling a longitudinal girder or a strip of unit width for obtaining longitudinal moments and shears due to live load. This beam is isolated from the rest of the structure and treated as a one-dimensional beam. This isolated beam is subjected to loads comprising one axle of the design vehicle multiplied by a load fraction “g.” This “g” is defined as Axle Load Distribution Factor in LRFD Specs., which is different from the Wheel Load Distribution Factor defined in the AASHTO Specs.

A. Moment -

(1) Interior — (Table 1-3 & AASHTO LRFD Table 4.6.2.2.2b-1)

(a) One lane

$$g_{\text{interior}} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

(b) Two or more lanes

$$g_{\text{interior}} = 0.75 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

$3'-6'' \le S \le 16'$, $4.5'' \le t_s \le 12''$, $20' \le L \le 240'$, $N_b \ge 4$

(2) Exterior — (Table 1-4 & AASHTO LRFD Table 4.6.2.2.2d-1)

(a) One lane Use level rule.

(b) Two or more lanes,

$$g_{\text{exterior}} = e \times g_{\text{interior}}$$

$e = 0.77 + d_e / 9.1$ $\quad -1' - 0'' \le d_e \le 5' - 6''$

FIGURE 1-3 ILLUSTRATION OF THE G-VALUE METHOD

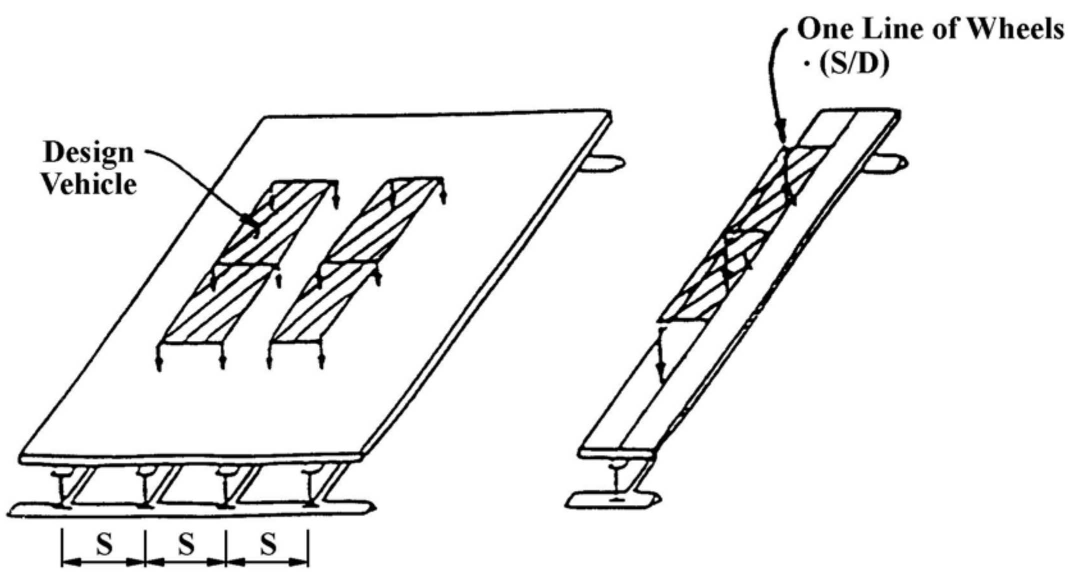


TABLE 1-3 AASHTO TABLE FOR THE MOMENT DISTRIBUTION FACTOR OF THE INTERIOR BEAMS

(AASHTO LRFD Table 4.6.2.2.2b-1 – Distribution of Live loads Per Lane for Moment in interior Beams)

Type of Beams	Applicable Cross-Section from Table – 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Wood Deck on Wood or Steel Beams	a, l	See Table 4.6.2.2.2a-1	
Concrete Deck on Wood Beams	l	Once Design Land Loaded: S/12.0 Two or More Design Lanes Loaded: S/10.0	$S \leq 6.0$
Concrete Deck, Filled Grid, Partially Filled Grid or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	$3.5 \leq S \leq 16.0$ $4.5 \leq t_s \leq 12.0$ $20 \leq L \leq 240$ $N_b \geq 4$ $10{,}000 \leq K_g \leq 7{,}000{,}000$
		Use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule.	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75+\frac{S}{3.6}\right)\left(\frac{1}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45}$	$7.0 \leq S \leq 13.0$ $60 \leq L \leq 240$ $N_c \geq 3$ If $N_c > 8$ use $N_c = 8$
		Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{5.8}\right)\left(\frac{1}{L}\right)^{0.25}$	
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded: $\left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12.0L^2}\right)^{0.25}$ Two ore More Design Lanes Loaded: $\left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125}$	$6.0 \leq S \leq 18.0$ $20 \leq L \leq 140$ $18 \leq d \leq 65$ $N_b \geq 3$
		Use Lever Rule	$S > 18.0$

TABLE 1-3 AASHTO TABLE FOR THE MOMENT DISTRIBUTION FACTOR OF THE INTERIOR BEAMS

(Continued)

<table>
<tr><th>Type of Beams</th><th>Applicable Cross-Section from Table – 4.6.2.2.1-1</th><th>Distribution Factors</th><th>Range of Applicability</th></tr>
<tr><td rowspan="4">Concrete Beams Used in Multi-Beam Decks</td><td>f</td><td rowspan="2">One Design Lane Loaded:
$k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$
where: $k = 2.5\,(N_b)^{-0.2} \geq 1.5$
Two or More Design Lanes Loaded:
$k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12.0L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$</td><td rowspan="2">$35 \leq b \leq 60$
$20 \leq L \leq 120$
$5 \leq N_b \leq 20$</td></tr>
<tr><td>g
if sufficiently connected to act as a unit</td></tr>
<tr><td>h</td><td rowspan="2">Regardless of Number of Loaded Lanes:
S/D
where:
$C = K\,(W/L)$
$D = 11.5 - N_L + 1.4\,N_L\,(1 - 0.2C)^2$
when $C \leq 5$
$D = 11.5 - N_L$ when $C > 5$
$K = \sqrt{\frac{(1+\mu)I}{J}}$
for preliminary design, the following value of K may be used:

<u>Beam Type</u> — <u>K</u>
Nonvoided rectangular beams — 0.7
Rectangular beams with circular voids: — 0.8
Box section beams — 1.0
Channel beams — 2.2
T-Beam — 2.0
Double T-Beam — 2.0</td><td rowspan="2">Skew≤ 45°
$N_L \leq 6$</td></tr>
<tr><td>g, i, j
if connected only enough to prevent relative vertical displacement at the interface</td></tr>
<tr><td>Open Steel Gird Deck on Steel Beams</td><td>a</td><td>One Design Lane Loaded:
$S/7.5$ if $t_g < 4.0$ IN
$S/10.0$ if $t_g \geq 4.0$ IN
Two or More Design Lanes Loaded:
$S/8.0$ if $t_g < 4.0$ IN
$S/10.0$ if $t_g \geq 4.0$ IN</td><td>$S \leq 6.0$ FT

$S \leq 10.5$ FT</td></tr>
<tr><td>Concrete deck on Multiple Steel Box Girders</td><td>b, c</td><td>Regardless of Number of Loaded Lanes:
$0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$</td><td>$0.5 \leq \frac{N_L}{N_b} \leq 1.5$</td></tr>
</table>

TABLE 1-4 AASHTO TABLE FOR THE MOMENT DISTRIBUTION FACTOR OF THE EXTERIOR BEAMS (AASHTO LRFD Table 4.6.2.2.2d-1)

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Wood Deck on Wood or Steel Beams	a, l	Lever Rule	Lever Rule	N/A
Concrete Deck on Wood	l	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T and Double T sections	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $e = 0.77 + \dfrac{d_e}{9.1}$	$-1.0 \le d_e \le 5.5$
			Use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule.	$N_b = 3$
Cast-in-Place Concrete Multi Cell Box	d	$g = \dfrac{W_e}{14}$	$g = \dfrac{W_e}{14}$	$W_e \le S$
		Or the provisions for a whole-width design specified in Article 4.6.2.2.1		
Concrete Deck on Concrete Spread Box Beams	b, c	Lever Rule	$g = e\, g_{interior}$ $e = 0.97 + \dfrac{d_e}{28.5}$	$0 \le d_e \le 4.5$ $6.0 < S \le 18.0$
			Use Lever Rule	$S > 18.0$
Concrete Box Beams Used in Multi-Beam Decks	f, g	Lever Rule	$g = e\, g_{interior}$ $e = 1.04 + \dfrac{d_e}{25}$	$-1.0 \le d_e \le 2.0$
Concrete Beams other than Box Beams Used in Multi-Beam Decks	h	Lever Rule	Lever Rule	N/A
	i, j if connected only enough to prevent relative vertical displacement at the interface			
Open Steel Gird Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Girders	b, c	As specified in Table 4.6.2.2.2b-1		

(3) Reduction on skew supports. (Table 1-5 & AASHTO LRFD Table 4.6.2.2.2e-1)

$$R = 1 - c_1\,(\tan 2)^{1.5}$$

$$c_1 = 0.25\left(\frac{k_g}{12Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

if $2 < 30^o$ then $c_1 = 0.0$

$2 > 60^o$ use $2 = 60^o$

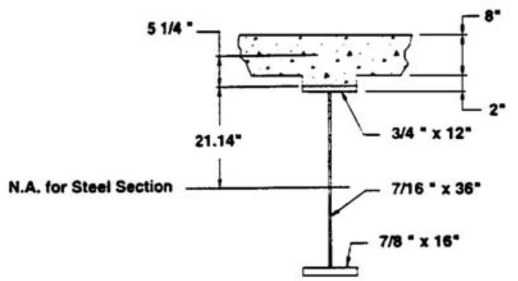
$30^o \le \theta \le 60^o$, $3.5' \le S \le 16.0'$, $20' \le L \le 240'$, $N_b \ge 4$

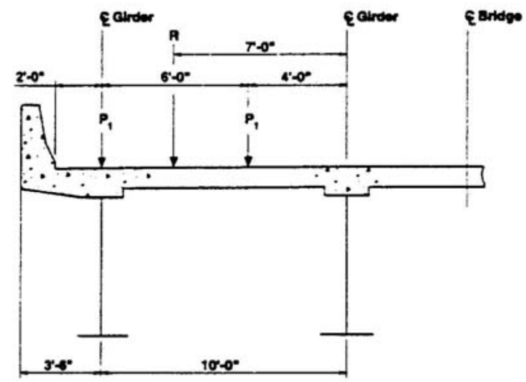
TABLE 1-5 AASHTO TABLE FOR THE MOMENT DISTRIBUTION FACTOR ON SKEWED SUPPORTS

(AASHTO LRFD Table 4.6.2.2.2e-1 – Reduction of Load Distribution Factors for Moment in Longitudinal Beams on Skewed Supports)

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Any Number of Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T or Double T Section	a, e, k i, j if sufficiently connected to act as a unit	$1 - c_1(\tan\theta)^{1.5}$ $c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$ If $\theta < 30^o$ then $c_1 = 0.0$ If $\theta > 60^o$ use $\theta = 60^o$	$30^o \le \theta \le 60^o$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$
Concrete Deck on concrete Spread Box Beams, Cast-in-Place Multicell Box Concrete Beams, and Double T Sections used in Multi-Beam Decks	b, c, d,f, g	$1.05 - 0.25\tan\theta \le 1.0$ if $\theta > 60^o$ use $\theta = 60^o$	$0 \le \theta \le 60^o$

FIGURE 1-4 EXAMPLE FOR THE CALCULATION OF MOMENT DISTRIBUTION FACTORS





Component	A	D	Ad	Ad²	I_o	I
Top Flange ¾″ × 12″	9.00	18.38	165.4	3,040	0.42	3,040
Web 7/16″ × 16″	15.75				1,701	1,701
Bottom Flange 7/8″ × 16″	14.00	-18.44	-258.2	4,760	0.89	4,761
	38.75		-92.8			9,502

$$-2.39(92.8) = -222$$

$$I_{NA} = 9{,}280 \text{ in.}^4$$

$$d_s = \frac{-92.8}{38.75} - 2.39 \text{ in.}$$

$d_{\text{TOP OF STEEL}}$ = 18.75 – 23.9 = 21.14 in.

$d_{\text{BOT OF STEEL}}$ = 18.88 – 2.39 = 16.49 in.

$$S_{\text{TOP OF STEEL}} = \frac{9{,}280}{21.14} = 439.0 \text{ in.}^3$$

$$S_{\text{BOT OF STEEL}} = \frac{9{,}280}{16.49} = 562.8 \text{ in.}^3$$

$$e_g = \frac{8.0}{2} + 2.0 + 21.14 - 0.75 = 26.39 \text{ in.} \qquad n = 8$$

$$K_g = n\left(I + Ae_g^2\right) = 8\left(9{,}280 + 38.75(26.39)^2\right) = 290{,}134 \text{ in.}^4$$

$$g_{\text{interior}} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1} = 0.484 \text{ lanes}$$

(for this case two lane loaded govens where g interior = 0.698 lanes

Exterior Girder – Strength Limit State

$$DF = \frac{7.0}{10.0} = 0.700 \qquad \text{(use the level rule for one lane loaded)}$$

Multiple presence factor m = 1.2 (Table 3.6.1.1.2-1)

1.2 (0.700) = 0.840 lanes

B. Shear -

(1) Interior — (Table 1-6 & AASHTO LRFD Table 4.6.2.2.3a-1)

(a) One lane $g_{interior} = 0.36 + S/25$

(b) Two or more lanes, $g_{interior} = 0.2 + S/12 - (S/35)^{2.0}$

$3'\text{-}5'' \le S \le 16'\text{-}0''$, $20' \le L \le 240'$, $4.5'' \le t_s \le 12.0''$, $N_b \ge 4$

(2) Exterior — (Table 1-7 & AASHTO LRFD Table 4.6.2.2.3b-1)

(a) One lane. Use level rule.

(b) Two or more lanes $g_{exterior} = e$ H $g_{interior}$

$e = 0.6 + d_e / 10$ $-1.0' \le d_e \le 5'\text{-}6''$

(3) Correction on the obtuse corner —

(Table 1-8 & AASHTO LRFD Table 4.6.2.2.3c-1)

$$g = 1 + 0.2\left(\frac{12Lt_s^3}{k_g}\right)^{0.3} \tan\theta$$

$0^\circ \le \theta \le 60^\circ$, $3'\text{-}5'' \le S \le 16'\text{-}0''$, $20' \le L \le 240'$, $N_b \ge 4$

TABLE 1-6 AASHTO TABLE FOR THE SHEAR DISTRIBUTION FACTOR OF THE INTERIOR BEAMS

(AASHTO LRFD Table 4.6.2.2.3a-1 – Distribution of Live Load Per Lane for Shear in Interior Beams)

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Wood Deck on Wood or Steel Beams	a, l	See Table 4.6.2.2.2a-1		
Concrete Deck on Wood Beams	l	Lever Rule	Lever Rule	N/A
Concrete Deck, filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.35+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{3}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $10{,}000 \le K_g \le 7{,}000{,}000$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	$\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1}$	$\left(\frac{S}{7.3}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1}$	$6.0 \le S \le 13.0$ $20 \le L \le 240$ $35 \le t_s \le 110$ $N_c \ge 3$
Concrete Deck on Concrete Spread Box Beams	b, c	$\left(\frac{S}{10}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1}$	$\left(\frac{S}{7.4}\right)^{0.8}\left(\frac{d}{12.0L}\right)^{0.1}$	$6.0 \le S \le 18.0$ $20 \le L \le 140$ $18 \le d \le 65$ $N_b \ge 3$
		Lever Rule	Lever Rule	$S > 18.0$
Concrete Box Beams Used in Multi-Beam Decks	f, g	$\left(\frac{b}{130L}\right)^{0.15}\left(\frac{I}{J}\right)^{0.05}$	$\left(\frac{b}{156}\right)^{0.4}\left(\frac{b}{12.0L}\right)^{0.1}\left(\frac{I}{J}\right)^{0.05}\left(\frac{b}{48}\right)$ $\frac{b}{48} > 1.0$	$35 \le b \le 60$ $20 \le L \le 120$ $5 \le N_b \le 20$ $25{,}000 \le J \le 610{,}000$ $40{,}000 \le I \le 610{,}000$
Concrete Beams Other Than Box Beams Used in Multi-Beam Decks	h i, j if connected only enough to prevent relative vertical displacement at the interface	Lever Rule	Lever Rule	N/A
Open Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Beams	b, c	As specified in Table 4.6.2.2.2b-1		

TABLE 1-7 AASHTO TABLE FOR THE SHEAR DISTRIBUTION FACTOR OF THE EXTERIOR BEAMS

(AASHTO LRFD Table 4.6.2.2.3b-1 – Distribution of Live Load Per Lane for Shear in Exterior Beams)

<table>
<tr><th>Type of Superstructure</th><th>Applicable Cross-Section from Table 4.6.2.2.1-1</th><th>One Design Lane Loaded</th><th>Two or more Design Lanes Loaded</th><th>Range of Applicability</th></tr>
<tr><td>Wood Deck on Wood or Steel Beams</td><td>a, l</td><td>Lever Rule</td><td>Lever Rule</td><td>N/A</td></tr>
<tr><td>Concrete Deck on Wood Beams</td><td>l</td><td>Lever Rule</td><td>Lever Rule</td><td>N/A</td></tr>
<tr><td rowspan="2">Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Beams</td><td rowspan="2">a, e, k and also i, j if sufficiently connected to act as a unit</td><td rowspan="2">Lever Rule</td><td>$g = e\, g_{interior}$
$e = 0.6 + \frac{d_e}{10}$</td><td>$-1.0 \le d_e \le 5.5$</td></tr>
<tr><td>Lever Rule</td><td>$N_b = 3$</td></tr>
<tr><td rowspan="2">Case-in-Place Concrete Multicell Box</td><td rowspan="2">d</td><td>Lever Rule</td><td>$g = e\, g_{interior}$
$e = 0.64 + \frac{d_e}{12.5}$</td><td rowspan="2">$-2.0 \le d_e \le 5.0$</td></tr>
<tr><td colspan="2">Or the provisions for a whole-width design specified in Article 4.6.2.2.1</td></tr>
<tr><td rowspan="2">Concrete Deck on Concrete Spread Box Beams</td><td rowspan="2">b, c</td><td rowspan="2">Lever Rule</td><td>$g = e\, g_{interior}$
$e = 0.8 + \frac{d_e}{10}$</td><td>$0 \le d_e \le 4.5$</td></tr>
<tr><td>Lever Rule</td><td>$S > 18.0$</td></tr>
<tr><td>Concrete Box Beams Used in Multi-Beam Decks</td><td>f, g</td><td>$g = e\, g_{interior}$
$e = 1.25 + \frac{d_e}{20} \ge 1.0$</td><td>$g = e\, g_{interior}$
$48/b \le 1.0$
$e = 1 + \left(\frac{d_e + \frac{b}{12} - 2.0}{40}\right)^{0.5} \ge 1.0$</td><td>$d_e \le 2.0$
35<b<60</td></tr>
<tr><td rowspan="2">Concrete Beams Other Than Box Beams Used in Multi-Beam Decks</td><td>h</td><td rowspan="2">Lever Rule</td><td rowspan="2">Lever Rule</td><td rowspan="2">N/A</td></tr>
<tr><td>i, j if connected only enough to prevent relative vertical displacement at the interface</td></tr>
<tr><td>Open Steel Grid Deck on Steel Beams</td><td>a</td><td>Lever Rule</td><td>Lever Rule</td><td>N/A</td></tr>
<tr><td>Concrete Deck on Multiple Steel Box Beams</td><td>b, c</td><td colspan="3">As specified in Table 4.6.2.2.2b-1</td></tr>
</table>

TABLE 1-8 AASHTO TABLE FOR THE DISTRIBUTION CORRECTION FACTOR FOR SUPPORT SHEAR OF THE OBTUSE CORNER

(AASHTO LRFD Table 4.6.2.2.3c-1 – Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner)

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T Section	a, e, k i, j if sufficiently connected to act as a unit	$1.0+0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$	$0^{o} \le \theta \le 60^{o}$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$
Cast-in-Place Concrete Multicell Box	d	$1.0+\left[0.25+\frac{12.0L}{70d}\right]\tan\theta$	$0^{o} \le \theta \le 60^{o}$ $6.0 \le S \le 13.0$ $20 \le L \le 240$ $35 \le d \le 110$ $N_c \ge 3$
Concrete Deck on Spread Concrete Box Beams	b, c	$1.0+\frac{\sqrt{\frac{Ld}{12.0}}}{6S}\tan\theta$	$0^{o} \le \theta \le 60^{o}$ $6.0 \le S \le 11.5$ $20 \le L \le 140$ $18 \le d \le 65$ $N_b \ge 3$
Concrete Box Beams Used in Multi-Beam Decks	f, g	$1.0+\frac{12.0L}{90d}\sqrt{\tan\theta}$	$0^{o} \le \theta \le 60^{o}$ $20 \le L \le 120$ $17 \le d \le 60$ $35 \le b \le 60$ $5 \le N_b \le 20$

2.0 STEEL STRUCTURES

2.1 Steel Material (LRFD Art. 6.4)

	AASHTO		Equiv. ASTM	
Structural Steel	M270	Grade 36	A709	Grade 36
		50		50
		50W		50W
		70W		70W
		100/100W		100/100W
Pins, Rollers	M169		A108	Grade 36
Rockers	M102		A668	Class C 33
				Class D 37.5
				Class F 50
				Class G 50
Bolts			A307	
	M164		A325	
	M253		A490	
Nuts	M291		A563	
Washers	M239		F436	
Studs	M169		A108	
Cap			A109	
Cast Steel	M192		A486	
	M103		A27	
	M163		A743	
Ductile Iron			A536	
Ferritic Malleable Iron Castings			A47	Grade 35018
Cast Iron Casting	M105	Class 30	A48	
Stainless Steel			A176	
			A240	
			A276	
			A666	
Wires (Cables)			A510	
			A641(Zinc-Coated)	
			A99 (Epoxy-Coated)	
			A603(Zinc-Coated Wire Rope)	
			A586(Zinc-Coated Parallel and Helical)	

.2 Fatigue and Fracture Limit State (LRFD Art. 6.6)

The fatigue provisions of the Steel Structures Section of the AASHTO LRFD Specification for Highway Bridge Design combine aspects of both the AASHTO Standard Specification for Highway Bridges (AASHTO 1996) and the Guide Specification for Fatigue Design of Steel Bridges (AASHTO 1989). These provisions are based upon two principles of fatigue of welded steel details:

- If all of the stress ranges that a welded steel detail experiences in its lifetime are less than the constant-amplitude fatigue threshold (i.e., the maximum stress range is less than the threshold), the detail will not experience fatigue crack growth; otherwise
- the fatigue life of the detail can be estimated considering an effective (weighted average of sorts) stress range, which represents all of the varying magnitudes of stress range experienced by the detail during its lifetime.

These two principles result in two branches in the flow of fatigue design, infinite life design and finite life design.

Fatigue details for bridges with higher truck traffic volumes are designed for infinite life. This practice is carried over from both the Standard Specifications and the Guide Specifications. Bridges with lower truck traffic volumes are designed for the fatigue life required by the estimated site-specific traffic volumes projected for their lifetimes.

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{(LRFD Eq. 6.6.1.2.2-1)}$$

(1) Infinite Fatigue Life (When the design stress range is less than one-half of the constant-amplitude fatigue threshold, the detail will theoretically provide infinite life.)

Detail Category	75-year $(ADTT)_{SL}$ equivalent to Infinite Life
A	535
B	865
B′	1035
C	1290
C′	745
D	1875
E	3545
E′	6525

(2) Finite Fatigue Life

$$Fn = \Delta F = \left(\frac{A}{N}\right)^{\frac{1}{3}} \geq \frac{1}{2}(\Delta F)_{TH}\ ; \qquad N = (365)(75)(n)(\mathrm{ADTT})_{\mathrm{single-lane}}$$

Category	A	$(\Delta F)_{TH}$ (ksi)		n	
A	2.5×10^{10}	24		P > 40′	P < 40′
B	1.2×10^{10}	16	Simple-span	1.0	2.0
B′	6.1×10^{9}	12			
C	4.4×10^{9}	10	Continuous		
C′	4.4×10^{9}	12	(1) Near Interior	1.5	2.0
D	2.2×10^{9}	7	Support		
E	1.1×10^{9}	4.5	(2) Elsewhere	1.0	2.0
E′	3.9×10^{8}	2.6			

Example: $(\mathrm{ADTT})_{\text{single-lane}}$ = 1500, 80′-80′ Continuous bridge.

Category C.

N = (365)(75)(1.5)(1500) = 6.159 H 10^7 - Interior support

(1.0) = 4.106 H 10^7 - elsewhere

F = $(4.4 \times 10^9/6.159 \times 10^7)^{1/3}$ = 4.15 ksi - Interior support

/4.106 = 4.75 ksi - elsewhere

½ $(\Delta F)_{TH}$ = ½ (10) = 5 ksi governs

Use F = 5 ksi

FIGURE 2-1 AASHTO STRESS RANGE VS NUMBER OF CYCLES

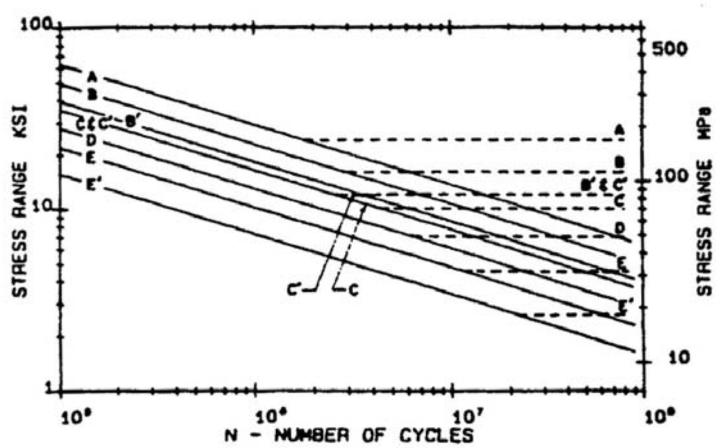
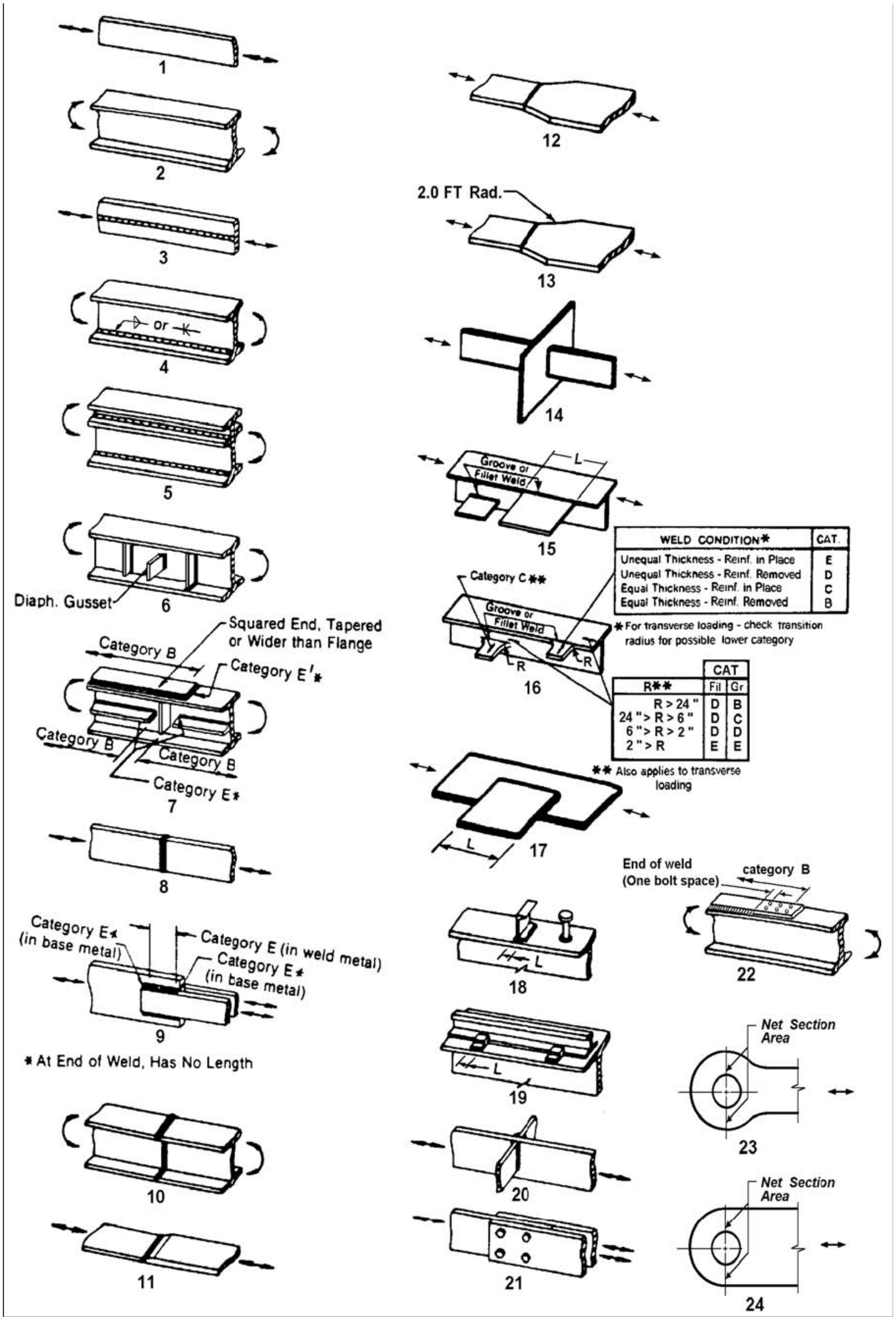


FIGURE 2-2 AASHTO ILLUSTRATIVE EXAMPLES FOR FATIGUE DETAILS



2.3 Resistance Factor (LRFD Art. 6.5.4.2)

For	flexure	ϕ_f	=	1.0
	shear	ϕ_v	=	1.0
	axial compression steel only	ϕ_c	=	0.9
	axial compression, composite	ϕ_c	=	0.9
	tension, fracture in net section	ϕ_u	=	0.80
	tension, yielding in gross section	ϕ_y	=	0.95

2.4 Tension Members (LRFD Art. 6.8)

(1) Axial Tension $P_r > \phi P_n$

where
$$\left.\begin{array}{l} P_r = \phi_y F_y A_g \\ P_r = \phi_u F_u A_n U \end{array}\right\} \text{ lesser}$$
(LRFD Eq. 6.8.2.1-1 & -2)

where the reduction factor, U, may be taken as:

— for sections subjected to a tension load transmitted directly to each of the cross sectional elements by bolts or welds

U = 1.0

— for bolted connections

- for rolled I-shapes with flange widths not less than 2/3 * depth, and structural tees cut from these shapes, connection is to the flanges and ≥ 3 fasteners U = 0.90

- for all other members and ≥ 3 fasteners U = 0.85

- for all members with 2 fasteners U = 0.75

— When a tension load is transmitted by fillet welds to some, but not all, elements of a cross-section, the weld strength shall control.

(2) Combined Axial Tension and Bending

— If $P_u / P_r < 0.2$ (LRFD Eq. 6.8.2.3-1)

$$\frac{P_u}{2P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0$$

— If $P_u / P_r \geq 0.2$ (LRFD Eq. 6.8.2.3-2)

$$\frac{P_u}{P_r} + \frac{8}{9} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0$$

(3) Limiting Slenderness Ratio (LRFD Art. 6.8.4)

— main members, subject to stress reversal $\ell / r \leq 140$

— main members, not subject to stress reversal $\ell / r \leq 200$

— bracing members $\ell / r \leq 240$

2.5 Compression Members (LRFD Art. 6.9)

Limitation of Plate -

— Slenderness of plates

$$\frac{b}{t} \leq k \sqrt{\frac{E}{F_y}}$$ (LRFD Eq. 6.9.4.2-1)

where k = 0.56 — Flanges and projecting legs or plates (one edge supported)

= 0.75 — Stems of rolled tees (one edge supported)

= 0.45 — Other projecting elements (one edge supported)

k = 1.4 — Box flanges and cover plates (two edges supported)

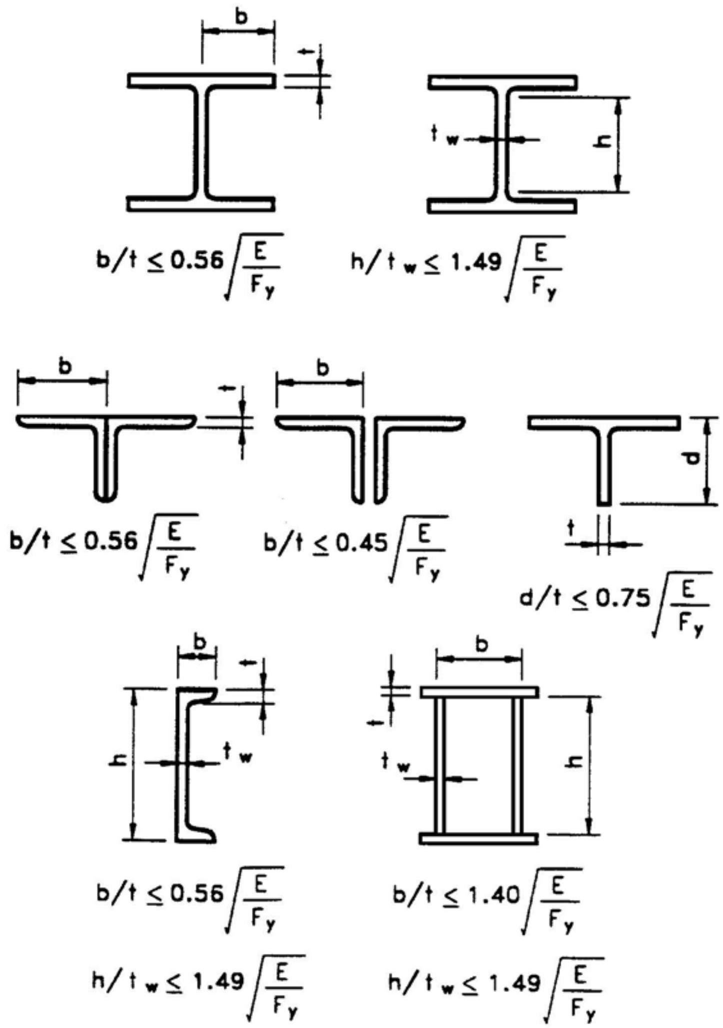
= 1.49 — Webs and other plate elements (two edges supported)

= 1.86 — Perforated cover plates (two edges supported)

— Wall thickness of tube

- circular $\frac{D}{t} \le 2.8\sqrt{\frac{E}{F_y}}$ (LRFD Eq. 6.9.4.2-5)
- rectangular $\frac{b}{t} \le 1.7\sqrt{\frac{E}{F_y}}$ (LRFD Eq. 6.9.4.2-6)

FIGURE 2-3 AASHTO LIMITING WIDTH-THICKNESS RATIOS



(1) Axial Compression $P_r = \phi\, P_n$

where $P_n = 0.66^{\lambda} F_y A_s$ for $\lambda \le 2.25$

(LRFD Eq. 6.9.4.1-1)

$$P_n = \frac{0.88 F_y A_s}{\lambda}$$ for $\lambda > 2.25$ (LRFD Eq. 6.9.4.1-2)

$$\lambda = \left(\frac{K\ell}{r_s \pi}\right)^2 \frac{F_y}{E}$$

For lateral support, in both directions, at their ends (LRFD Art. 4.6.2.5)

K	=	0.75	for bolted or welded end
	=	0.875	for pinned ends
	=	1.0	For single angles, regardless of end connection

(2) Combined Axial Compression and Bending (LRFD Art. 6.9.2.2)

— If $P_u / P_r < 0.2$

$$\frac{P_u}{2P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ (LRFD Eq. 6.9.2.2-1)

— If $P_u / P_r \ge 0.2$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ (LRFD Eq. 6.9.2.2-2)

(3) Limiting Slenderness Ratio (LRFD Art. 6.9.3)

— main members $K\ell / r \le 120$

— bracing members $K\ell / r \le 140$

(4) Composite column and Axial compression

$$P_n = 0.66^{\lambda} F_e A_s \quad \text{for} \quad \lambda \le 2.25 \qquad \text{(LRFD Eq. 6.9.5.1-1)}$$

$$P_n = \frac{0.88 F_e A_s}{\lambda} \quad \text{for} \quad \lambda > 2.25 \qquad \text{(LRFD Eq. 6.9.5.1-2)}$$

$$\lambda = \left(\frac{K\ell}{r_s \pi}\right)^2 \frac{F_e}{E_e} \qquad \text{(LRFD Eq. 6.9.5.1-3)}$$

$$F_e = F_y + C_1 F_{yr}\left(\frac{A_r}{A_s}\right) + C_2 f_c'\left(\frac{A_c}{A_s}\right) \qquad \text{(LRFD Eq. 6.9.5.1-4)}$$

$$E_e = E\left[1 + \left(\frac{C_3}{n}\right)\left(\frac{A_c}{A_s}\right)\right] \qquad \text{(LRFD Eq. 6.9.5.1-5)}$$

Column Type	C_1	C_2	C_3
Concrete Filled tubing	1.0	0.85	0.4
Concrete encased shape	0.7	0.6	0.2

2.6 I-Section Flexural Members

2.6.1 Composite Sections

(1) Effective Width (LRFD Art. 4.6.2.6)

~~(a) Interior - min. of~~ $\begin{cases} \frac{1}{4} L_{eff} \\ 12t_{slab} + \max.of\,(\frac{1}{2} t_{web} \; and \; \frac{1}{2} w_{topflange}) \\ average \;\; spacing \;\; of \;\; adjacent \;\; beams \end{cases}$

~~(b) Exterior - min. of~~ $\begin{cases} \frac{1}{8} L_{eff} \\ 6t_{slab} + \max.of\,(\frac{1}{2} t_{web} \; and \; \frac{1}{4} w_{topflange}) \\ width \;\; of \;\; the \;\; overhang \end{cases}$

2008: Interior - one-half the distance to the adjacent girder on each side of the component;

Exterior – one-half the distance to the adjacent girder plus the full overhang width.

(2) Yield Moment Resistance $M_y = M_{D1} + M_{D2} + M_{AD}$

Solve for the M_{AD} from

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{DL}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{(LRFD D6.2.2-1)}$$

S_{NC}	=	Non-composite section modulus
S_{ST}	=	Short-term composite section modulus
S_{LT}	=	Long-term composite section modulus
M_{D1}, M_{D2} & M_{AD}	=	Moments due to the factored loads

(3) Depth of Web in Compression

– Elastic (D_c)

- Positive flexure (distance from web top to elastic neutral axis)

$$D_c = \left[\frac{|f_C|}{|f_C| + f_t} \right] d - t_f \qquad \text{(LRFD D 6.3.1-1)}$$

- Negative flexure (distance from web bottom to elastic neutral axis)

  D_c may be computed for the section consisting of the steel girders plus the longitudinal reinforcement.

– Plastic (D_{cp}), Positive flexure (distance from web top to plastic neutral axis)

- The plastic natural axis is in the web.

$$D_{cp} = \frac{D}{2}\left[\frac{F_{yt}A_t - F_{yc}A_c - 0.85f_c'A_s - F_{yr}A_r}{F_{yw}A_w} + 1\right] \qquad \text{(LRFD D 6.3.2-1)}$$

- All others, $D_{CP} = 0$

– Plastic (D_{cp}), Negative flexure (distance from web bottom to plastic neutral axis)

- The plastic natural axis is in the web

$$D_{cp} = \frac{D}{2A_wF_{yw}}\left(F_{yt}A_t + F_{yw}A_w + F_{yr}A_r - F_{yc}A_c\right) \qquad \text{(LRFD D 6.3.2-2)}$$

- All others, $D_{CP} = D$

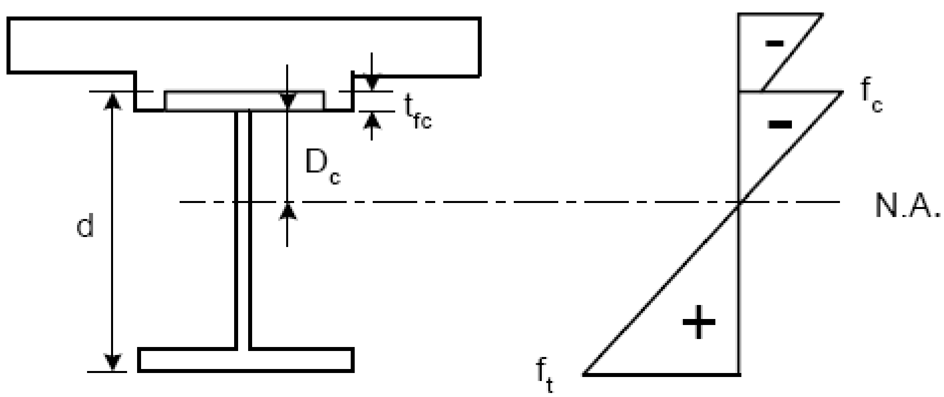


Figure 2-4 Computation of Dc at sections in Positive Flexure

TABLE 2-1 AASHTO TABLE OF THE PLASTIC MOMENT FOR THE POSITIVE BENDING SECTIONS

(AASHTO LRFD Table D6.1-1 – Calculation of $\bar{y}$ and M_p for Positive ending Sections)

CASE	PNA	CONDITION	$\bar{y}$ AND M_p
I	In Web	$P_t + P_w \geq P_c + P_s + P_{rb} + P_{rt}$	$\bar{y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_p = \frac{P_w}{2D}\left[\bar{y}^2 + \left(D - \bar{y}\right)^2\right] + \left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t\right]$
II	In Top Flange	$P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$	$\bar{y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$ $M_p = \frac{P_c}{2t_c}\left[\bar{y}^2 + \left(t_c - \bar{y}\right)^2\right] + \left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t\right]$
III	Slab, Below P_{rb}	$P_t + P_w + P_c \geq \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rb} + P_{rt}$	$\bar{y} = \left(t_s\right)\left[\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right]$ $M_p = \left(\frac{\bar{y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
IV	Slab, at P_{rb}	$P_t + P_w + P_c + P_{rb} \geq \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rt}$	$\bar{y} = C_{rb}$ $M_p = \left(\frac{\bar{y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_c d_c + P_w d_w + P_t d_t\right]$
V	Slab, Above $P_{rb,}$ Below P_{rt}	$Pt + P_w + P_c + P_{rb} + P_{rt} \geq \left(\frac{C_{rt}}{t_s}\right) P_s$	$\bar{y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
VI	Slab, at P_{rt}	$Pt + P_w + P_c + P_{rb} \geq \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rt}$	$\bar{y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
VII	Slab, above P_{rt}	$Pt + P_w + P_c + P_{rb} < \left(\frac{C_{rt}}{t_s}\right) P_s + P_{rt}$	$\bar{y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$

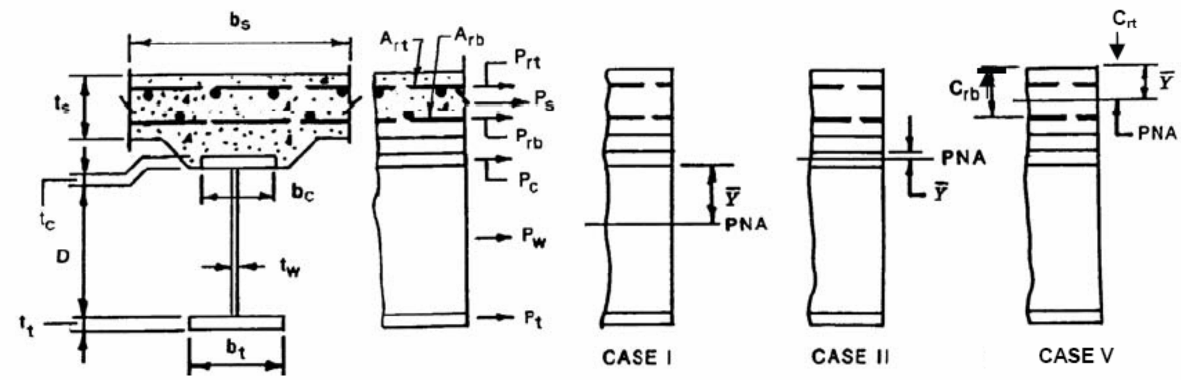
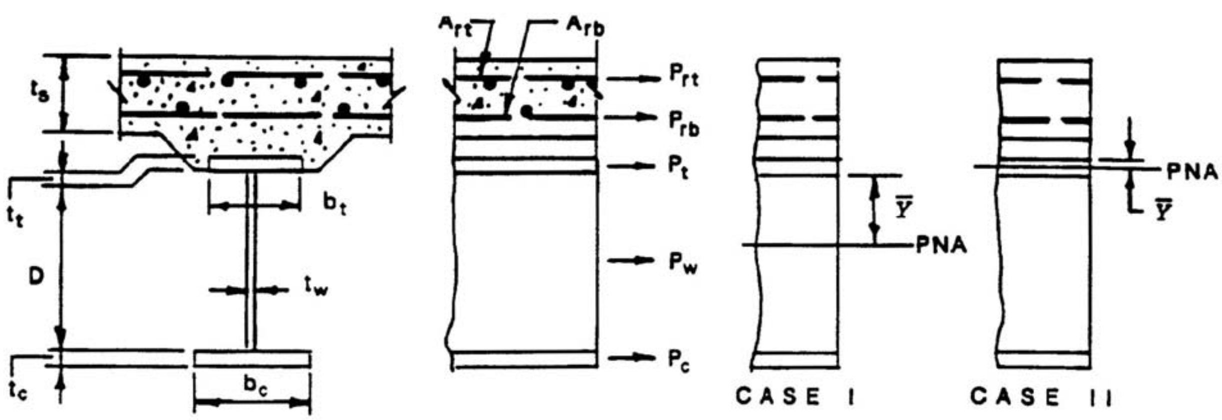


TABLE 2-2 AASHTO TABLE OF THE PLASTIC MOMENT FOR THE NEGATIVE BENDING SECTIONS

(AASHTO LRFD Table D6.1-2 – Calculation of $\bar{y}$ and M_p for Negative Bending Sections)

CASE	PNA	CONDITION	$\bar{y}$ and M_p
I	In Web	$P_c + P_w \geq P_t + P_{rb} + P_{rt}$	$\bar{y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_p = \frac{P_w}{2D}\left[\bar{y}^2 + \left(D - \bar{y}\right)^2\right] + \left[P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c\right]$
II	In Top Flange	$P_c + P_w + P_t \geq P_{rb} + P_{rt}$	$\bar{y} = \left(\frac{t_t}{2}\right)\left[\frac{P_w + P_c - P_{rt} - P_{rb}}{P_t} + 1\right]$ $M_p = \frac{P_t}{2t_t}\left[\bar{y}^2 + \left(t_t - \bar{y}\right)^2\right] + \left[P_{rt}d_{rt} + P_{rb}d_{rb} + P_w d_w + P_c d_c\right]$



2.6.2 Noncomposite Sections

Sections where the concrete deck is not connected to the steel section by shear connectors designed in this section shall be considered noncomposite sections.

Depth of web in compression for plastic:

If: $F_{yw}A_w \geq |F_{yc}A_c - F_{yt}A_t|$ Then

$$D_{cp} = \frac{D}{2A_w F_{yw}}\left(F_{yt}A_t + F_{yw}A_w - F_{yc}A_c\right) \qquad \text{(LRFD Eq. D 6.3.2-4)}$$

Otherwise

$$D_{cp} = D$$

2.7 Cross-Section Proportion Limits (LRFD Art. 6.10.2)

<table>
<tr><td rowspan="2">Web Proportions</td><td>w/o longitudinal Stiffeners</td><td>$\frac{D}{t_w} \leq 150$ (LRFD Eq. D 6.10.2.1.1-1)</td></tr>
<tr><td>w longitudinal Stiffeners</td><td>$\frac{D}{t_w} \leq 300$ (LRFD Eq. D 6.10.2.1.2-1)</td></tr>
<tr><td rowspan="2">Flange Proportions</td><td>Compression Flanges</td><td rowspan="2">$\frac{b_f}{2t_f} \leq 12.0$ (LRFD Eq. D 6.10.2.2-1)
$b_f \geq D/6$ (LRFD Eq. D 6.10.2.2-2)
$t_f \geq 1.1t_w$ (LRFD Eq. D 6.10.2.2-3)
$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$ (LRFD Eq. D 6.10.2.2-4)</td></tr>
<tr><td>Tension Flanges</td></tr>
</table>

2.8 Constructibility (LRFD Art. 6.10.3)

(1) Flexural Requirement

Discretely Braced Flanges in Compression	$f_{bu} + f_{\ell} \le \phi_f R_h F_{yc}$ (LRFD Eq. 6.10.3.2.1-1)	For sections with slender webs, it shall not be checked when f_{ℓ} is equal to zero.
	$f_{bu} + \frac{1}{3} f_{\ell} \le \phi_f R_h F_{yc}$ (LRFD Eq. 6.10.3.2.1-2)	
	$f_{bu} \le \phi_f F_{crw}$ (LRFD Eq. 6.10.3.2.1-3)	For sections with compact or noncompact webs, It shall not be checked.
Discretely Braced Flanges in Tension	$f_{bu} + f_{\ell} \le \phi_f R_h F_{yt}$ (LRFD Eq. 6.10.3.2.2-1)	
Continuously Braced Flanges in Tension or Compression	$f_{bu} \le \phi_f R_h F_{yf}$ (LRFD Eq. 6.10.3.2.3-1)	
Concrete Deck	The longitudinal tensile stress in a composite concrete deck due to the factored loads shall not exceed ϕf_r during critical stages of construction, unless longitudinal reinforcement is provided according to the provisions of LRFD Article 6.10.1.7.	f_r shall be taken as the modulus of rupture of the concrete determined as specified in LRFDArticle 5.4.2.6

(2) Shear Requirement

Interior panels of webs with transverse stiffeners, with or without longitudinal stiffeners, shall satisfy the following requirement during critical stages of construction:

$$V_u \le \phi_v V_{cr} \quad \text{(LRFD Eq. 6.10.3. 3-1)}$$

2.9 Service Limit State (Permanent Deformations) (LRFD Art. 6.10.4.2)

(1) For Composite:

a) For the top steel flange:

$$f_f \leq 0.95 R_h F_{yf}$$ (LRFD Eq. 6.10.4.2.2-1)

b) For the bottom steel flange:

$$f_f + 0.5 f_\ell \leq 0.95 R_h F_{yf}$$ (LRFD Eq. 6.10.4.2.2-2)

(2) For <u>Noncomposite</u>:

$$f_f + 0.5 f_\ell \leq 0.80 R_h F_{yf}$$ (LRFD Eq. 6.10.4.2.2-3)

Except for composite sections in positive flexure, all sections shall satisfy:

$$f_c \leq F_{crw}$$ (LRFD Eq. 6.10.4.2.2-4)

2.10 Fatigue and Fracture Limit State (LRFD Art. 6.10.5.3)

— The Fatigue load combination the fatigue live load shall follow Section 2.1.4 (LRFD Art. 3.6.1.4)

— The provisions for fatigue in shear connectors shall follow Section 2.2.15 (LRFD Art. 6.10.10)

— Special Fatigue Requirement for Webs

Interior panels of webs w/ transverse stiffeners, w/ or w/o longitudinal stiffeners:

$$V_u \leq V_{cr}$$ (LRFD Eq. 6.10.5.3-1)

2.11 Strength Limit State (LRFD Art. 6.10.6)

2.11.1 Flexure

(1) General

If there are holes in the tension flange, the tension flange shall satisfy:

$$f_t \leq 0.84 \left(\frac{A_n}{A_g} \right) F_u \leq F_{yt}$$ (LRFD Eq. 6.10.1.8-1)

(2) Composite Sections in Positive Flexure

a) Composite sections in straight bridges that satisfy the following requirements shall qualify as compact composite sections:

- the specified minimum yield strengths of the flanges do not exceed 70.0 ksi,
- the web satisfies the requirement of Section 2.2.7 (LRFD Art. 6.10.2)
- the section satisfies the web slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} \qquad \text{(LRFD Eq. 6.10.6.2.2-1)}$$

b) Compact and Noncompact sections shall satisfy the requirements of Section 2.2.12 (LRFD Art. 6.10.7).

(3) Composite Sections in Negative Flexure and Noncomposite Sections

Sections in kinked (chorded) continuous or horizontally curved steel girder bridges shall be proportioned according to provisions specified in Section 2.2.13 (LRFD Art. 6.10.8)

2.11.2 Shear

Follow Section 2.2.14 (LRFD Art. 6.10.9)

2.11.3 Shear Connector

Follow Section 2.2.15 (LRFD Art. 6.10.10)

2.12 Flexural Resistance-Composite Sections in Positive Flexure (LRFD Art. 6.10.7)

(1) Compact Sections:

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_n \qquad \text{(LRFD Eq. 6.10.7.1.1-1)}$$

If $D_p \leq 0.1 D_t$, $\quad M_n = M_p$ (LRFD Eq. 6.10.7.1.2-1)

Otherwise, $\quad M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)$ (LRFD Eq. 6.10.7.1.2-2)

In a continuous span, $\quad M_n \leq 1.3 R_h M_y$ (LRFD Eq. 6.10.7.1.2-3)

Unless:

- the span under consideration and all adjacent interior-pier sections satisfy the requirements of LRFD Article B6.2,
- the appropriate value of θ_{RL} from LRFD Article B6.6.2 exceeds 0.009 radians at all adjacent interior-pier sections.

(2) Noncompact Sections

- Compression flange: $\quad f_{bu} \leq \phi_f F_{nc}$ (LRFD Eq. 6.10.7.2.1-1) where $F_{nc} = R_b R_h F_{yc}$ (LRFD Eq. 6.10.7.2.2-1)

- Tension flange: $f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nt}$ (LRFD Eq. 6.10.7.2.1-2)

  where $F_{nc} = R_h F_{yt}$ (LRFD Eq. 6.10.7.2.2-2)

- For shored construction,

  the maximum longitudinal compressive stress in the concrete deck $\le 0.6 f_c'$

(3) Ductility Requirement

$$D_p \le 0.42 D_t$$ (LRFD Eq. 6.10.7.3-1)

2.13 Composite Sections in Negative Flexure and Noncomposite Sections (LRFD Art. 6.10.8)

(1) General

Discretely Braced Flanges in Compression	$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc}$ (LRFD Eq. 6.10.8.1.1-1)
Discretely Braced Flanges in Tension	$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nt}$ (LRFD Eq. 6.10.8.1.2-1)
Continuously Braced Flanges in Tension or Compression	$f_{bu} \le \phi_f R_h F_{yf}$ (LRFD Eq. 6.10.8.1.3-1)

(2) Compression-Flange Flexural Resistance

Local Buckling (FLB) Resistance	$\lambda_f \le \lambda_{pf}$	$F_{nc} = R_b R_h F_{yc}$
	otherwise	$F_{nc} = \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc}$
Lateral Torsional Buckling (LTB) Resistance	$L_b \le L_p$	$F_{nc} = R_b R_h F_{yc}$
	$L_p < L_b \le L_r$	$F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \le R_b R_h F_{yc}$
	$L_b > L_r$	$F_{nc} \le R_b R_h F_{yc}$

(3) Tension-Flange Flexural Resistance

$$F_{nt} = R_h F_{yt} \qquad \text{(LRFD Eq. 6.10.8.3-1)}$$

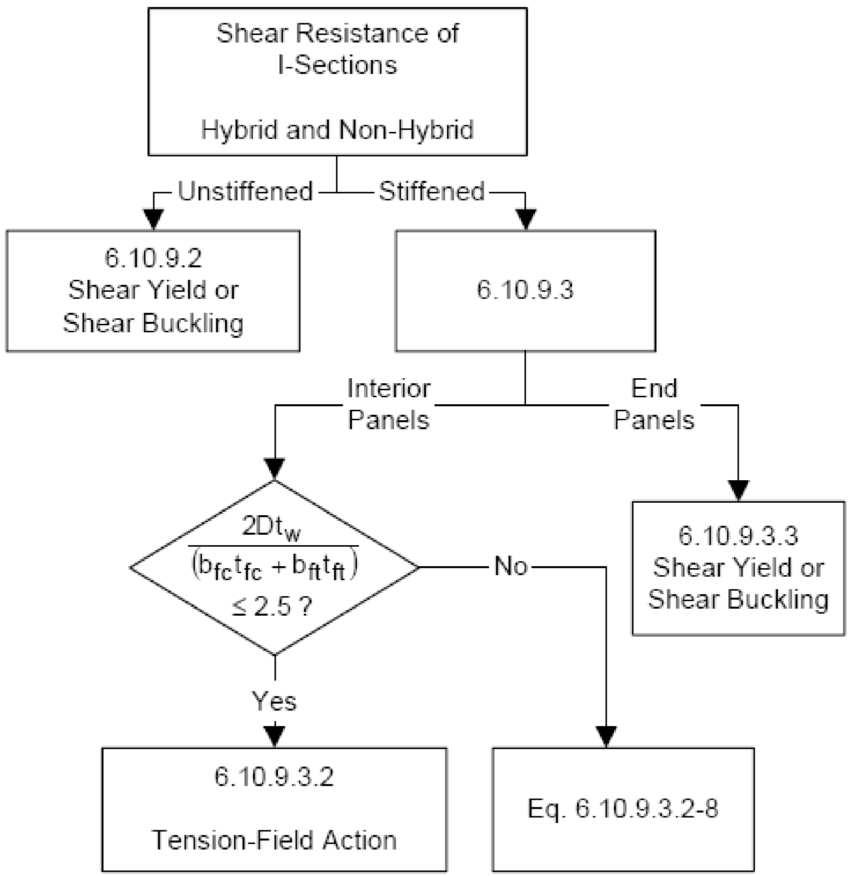
2.14 Shear Resistance (LRFD Art. 6.10.9)

At the strength limit state, straight and curved web panel shall satisfy:

$$V_u \le \phi_v V_n \qquad \text{(LRFD Eq. 6.10.9.1-1)}$$

A flowchart for determining the shear resistance of I-section is shown in Figure 2-9 (also AASHTO LRFD Figure C6.10.9.1-1)

Figure 2-5 Flowchart for Shear Design of I Sections



(1) Unstiffened web (LRFD Art. 6.10.9.2)

$$V_n = CV_p \qquad \text{(LRFD Eq. 6.10.9.2-1)}$$

$$V_p = 0.58\, F_{yw}\, Dt_w \qquad \text{(LRFD Eq. 6.10.9.2-2)}$$

(2) Stiffened web

a) Interior Panels — (LRFD Art. 6.10.9.3.2)

— if $\frac{2Dt_w}{\left(b_{fc}t_{fc}+b_{ft}t_{ft}\right)} \le 2.5$:

$$V_n = V_p\left[C+\frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \qquad \text{(LRFD Eq. 6.10.9.3.2-2)}$$

— otherwise:

$$V_n = V_p\left[C+\frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}+\frac{d_o}{D}}\right] \qquad \text{(LRFD Eq. 6.10.9.3.2-8)}$$

for which

$V_p = 0.58\ F_{yw}\ D\ t_w$ (LRFD Eq. 6.10.9.3.2-3)

- Determination of C

— if $\frac{D}{t_w} < 1.12\sqrt{\frac{Ek}{F_{yw}}}$

C = 1.0 (LRFD Eq. 6.10.9.3.2-4)

— if $1.12\sqrt{\frac{Ek}{F_{yw}}} \le \frac{D}{t_w} \le 1.40\sqrt{\frac{Ek}{F_{yw}}}$

$$C = \frac{1.12}{\frac{D}{t_w}} \sqrt{\frac{Ek}{F_{yw}}}$$ (LRFD Eq. 6.10.9.3.2-5)

— if $\frac{D}{t_w} > 1.40 \sqrt{\frac{Ek}{F_{yw}}}$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \left(\frac{Ek}{F_{yw}}\right)$$ (LRFD Eq. 6.10.9.3.2-6)

where $k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2}$ (LRFD Eq. 6.10.9.3.2-7)

b) End Panels — (LRFD Art. 6.10.9.3.3)

$V_n = C\, V_p$ (LRFD Eq. 6.10.9.3.3-1)

where $V_p = 0.58\, F_{yw}\, D\, t_w$ (LRFD Eq. 6.10.9.3.3-2)

- w/o longitudinal stiffener: $d_o/D \leq 1.5$
- w/ longitudinal stiffener: $d_o/D \leq 1.5$

2.15 Shear Connectors (LRFD Art. 6.10.10)

- In the negative flexure regions, shear connectors shall be provided where the longitudinal reinforcement is considered to be a part of the composite section.
- Otherwise, shear connectors need not be provided in negative flexure regions, but additional connectors shall be placed in the region of the points of permanent load contraflexure.

$$n_{AC} = \frac{A_r f_{sr}}{Z_r}$$ (LRFD Eq. 6.10.10. 3-1)

(1) Fatigue Limit State

$$p \le \frac{nZ_r}{V_{sr}}$$ (LRFD Eq. 6.10.10.1.2-1)

$Z_r = \alpha\ d^2 \geqslant 5.5\ d^2/2;$ (LRFD Eq. 6.10.10. 2-1)

where $\alpha = 34.5 - 4.28 \log N$ (LRFD Eq. 6.10.10. 2-2)

(2) Strength Limit State

$$Q_r = \phi_{sc} Q_n$$ (LRFD Eq. 6.10.10.4.1-1)

$$n = \frac{P}{Q_r}$$ (LRFD Eq. 6.10.10.4.1-2)

(a) Nominal Shear Force,

- Simple&continuous spans that are noncomposite for negative flexure:

$$P = \sqrt{P_p^2 + F_P^2}$$ (LRFD Eq. 6.10.10.4.2-1)

where

$$P_p = \min.\ \ of \begin{cases} 0.85 f'_c b_s t_s & \text{(LRFD Eq. 6.10.10.4.2 - 2)} \\ F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} & \text{(LRFD Eq. 6.10.10.4.2 - 3)} \end{cases}$$

$$F_p = P_p \frac{L_p}{R}$$ (LRFD Eq. 6.10.10.4.2-4)

(For straight spans or segments, F_P may be taken equal to zero)

- Continuous spans that are composite for negative flexure:

$$P = \sqrt{P_T^2 + F_T^2}$$ (LRFD Eq. 6.10.10.4.2-5)

where

$$P_T = P_p + P_n \qquad \text{(LRFD Eq. 6.10.10.4.2-6)}$$

$$P_n = \min. \quad of \quad \begin{cases} F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} & \text{(LRFD Eq. 6.10.10.4.2 - 7)} \\ 0.45 f'_c b_s t_s & \text{(LRFD Eq. 6.10.10.4.2 - 8)} \end{cases}$$

$$F_T = P_T \frac{L_n}{R} \qquad \text{(LRFD Eq. 6.10.10.4.2-9)}$$

(b) Shear Resistance, Q_n

- Stud shear connector

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u \qquad \text{(LRFD Eq. 6.10.10.4.3-1)}$$

- Channel shear connector

$$Q_n = 0.3 (t_f + 0.5 t_w) L_c \sqrt{f_c' E_c} \qquad \text{(LRFD Eq. 6.10.10.4.3-2)}$$

2.16 Transverse Stiffeners (LRFD Art. 6.10.11.1)

- Stiffeners in straight girders not used as connection plates shall be tight fit at the compression flange, but need not be in bearing with the tension flange.
- Stiffeners used as connecting plates for diaphragms or cross-frames shall be attached to both flanges.

The width, b_t, of each projecting stiffener element shall satisfy:

$$b_t \geq 2.0 + \frac{D}{30.0} \qquad \text{(LRFD Eq. 6.10.11.1.2-1)}$$

and

$$16.0\, t_p \geq b_t \geq 0.25\, b_f \qquad \text{(LRFD Eq. 6.10.11.1.2-2)}$$

where : b_f = full-width of steel flange

The moment of inertia of any transverse stiffener must satisfy:

$$\min. \quad of \quad \begin{cases} I_t \geq b t_w^3 J \\ I_t \geq \dfrac{D^4 \rho_t^{1.3}}{40} \left(\dfrac{F_{yw}}{E} \right)^{1.5} \end{cases} \qquad \text{(LRFD Eq.6.10.11.1.3-1\&2)}$$

for which:

$$J = 2.5\left(\frac{D}{d_o / D} \right)^2 - 2.0 \geq 0.5 \qquad \text{(LRFD Eq.6.10.11.1.3-3)}$$

where:

I_t	=	moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs
b	=	the smaller of d_o and D
t_w	=	web thickness
d_o	=	the smaller of the adjacent web panel widths
D	=	web depth

Transverse stiffeners used in web panels with longitudinal stiffeners must also satisfy:

$$I_t \geq \left(\frac{b_t}{b_\ell} \right) \left(\frac{D}{3.0 d_o} \right) I_\ell \qquad \text{(LRFD Eq.6.10.11.1.3-5)}$$

where:

b_t	=	projecting width of transverse stiffener
b_ℓ	=	projecting width of longitudinal stiffener
I_ℓ	=	moment of inertia of the longitudinal stiffener determined by (LRFD Eq.6.10.11.3.3-1)
D	=	web depth

2.17 Bearing Stiffeners (LRFD Art. 6.10.11.2)

Bearing stiffeners should be placed on webs of builtup sections at all bearing locations.

- Bearing stiffeners should be placed on the webs of plate girders at all bearing locations and at all locations supporting concentrated loads.
- Bearing stiffeners consist of one or more plates or angles welded or bolted to both sides of the web. The connections to the web are to be designed to transmit the full bearing force due to the factored loads.
- The stiffeners should extend the full-depth of the web and, as closely as practical, to the outer edges of the flanges.

The width, b_t, of each projecting stiffener element must satisfy:

$$b_t \le 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$ (LRFD Eq.6.10.11.2.2-1)

where:

t_p = thickness of projecting element

F_{ys} = specified minimum yield strength of the stiffener

The factored bearing resistance, $(R_{sb})_r$, shall be taken as:

$$(R_{sb})_r = \phi_b (R_{sb})_n = 1.4\phi_b A_{pn} F_{ys}$$ (LRFD Eq.6.10.11.2.3-1)

where:

F_{ys} = specified minimum yield strength of the stiffener

A_{pn} = area of the projecting elements of the stiffener outside of the web-to-flange fillet welds, but not beyond the edge of the flange

ϕ_b = resistance factor for bearing

2.18 Longitudinal Stiffeners (LRFD Art. 6.10.11.3)

Where required, longitudinal stiffeners should consist of either a plate welded to one side of the web, or a bolted angle. Longitudinal stiffeners shall be located at a vertical position on the web such that constructability (LRFD Eq. 6.10.3.2.1-3) is satisfied, requirement (LRFD Eq. 6.10.4.2.2-4) is satisfied at the service limit state, and all the appropriate design requirements are satisfied at the strength limit state.

The flexural stress in the longitudinal stiffener, f_s, due to the factored loads at the strength limit state and when checking constructability shall satisfy:

$$f_s \le \phi_f R_h F_{ys} \qquad \text{(LRFD Eq.6.10.11.3.1-1)}$$

The projecting width, b_ℓ, of the stiffener must satisfy:

$$b_\ell \le 0.48 t_s \sqrt{\frac{E}{F_{ys}}} \qquad \text{(LRFD Eq.6.10.11.3.2-1)}$$

where:

t_s = thickness of stiffener

F_{ys} = specified minimum yield strength of the stiffener

Longitudinal stiffeners must satisfy:

$$I_\ell \ge D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \beta \qquad \text{(LRFD Eq.6.10.11.3.3-1)}$$

$$r \ge \frac{0.16 d_o \sqrt{\frac{F_{ys}}{E}}}{\sqrt{1 - 0.6 \frac{F_{yc}}{R_h F_{ys}}}} \qquad \text{(LRFD Eq.6.10.11.3.3-2)}$$

where:

β = curvature correction factor for longitudinal stiffener rigidity

<table>
<tr><th>β</th><th>Case</th><td rowspan="3">Z: curvature parameter
$Z = \frac{0.95 d_0^2}{R t_w} \le 10$</td></tr>
<tr><td>$\frac{Z}{6} + 1$</td><td>the longitudinal stiffener is on the side of the web away from the center of curvature</td></tr>
<tr><td>$\frac{Z}{12} + 1$</td><td>the longitudinal stiffener is on the side of the web toward the center of curvature</td></tr>
</table>

I_ℓ = moment of inertia of the longitudinal stiffener including an effective width of the web equal to $18 t_w$ taken about the neutral axis of the combined section

r = radius of gyration of the longitudinal stiffener including an effective width of the web equal to $18 t_w$ taken about the neutral axis of the combined section

D = web depth

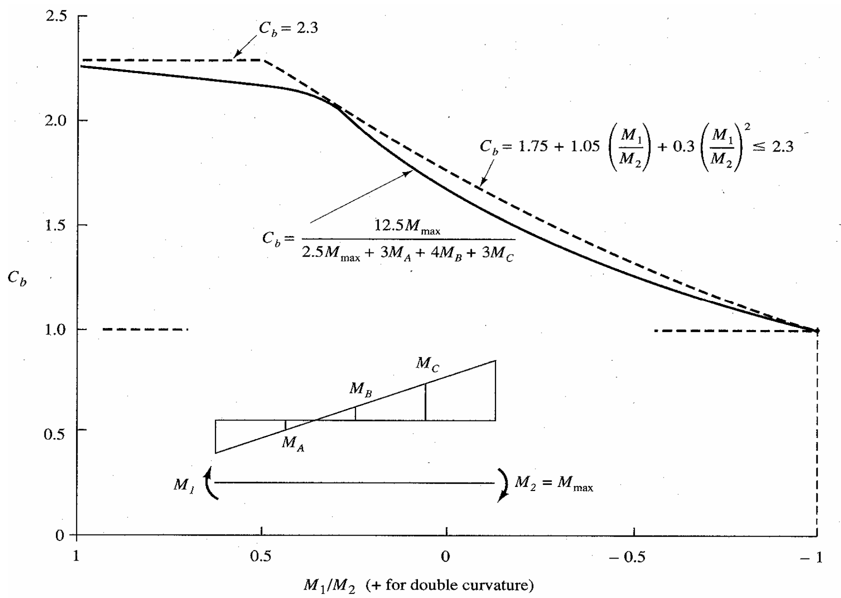
d_o = transverse stiffener spacing

t_w = web thickness

F_{ys} = specified minimum yield strength of the stiffener

A longitudinal stiffener meeting the requirements above will have sufficient area to anchor the tension field. Therefore, no additional area requirement is given for longitudinal stiffeners.

Appendix - Modification Factor C_b (for non-uniform bending moment variation)

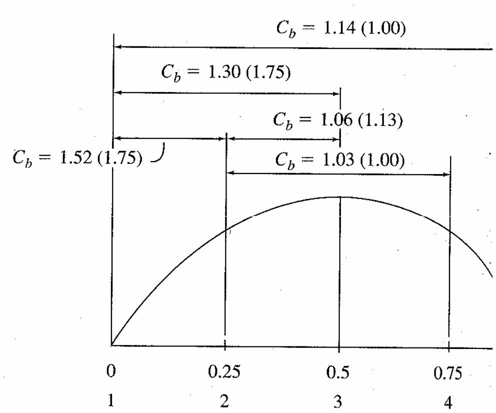


C_b for a Simple Span Bridge

C_b FOR PARABOLIC SEGMENTS USING LRFD-F1.2a, FORMULA (C-F1-3), EQ. 9.6.11*

Case 1	Laterally braced at ends; points 1 and 5 only; M_{max} at 3	$C_b = 1.14$
Case 2	Laterally braced at ends and midspan; points 1,3, and 5 only; M_{max} at 3	$C_b = 1.30$
Case 3	Laterally braced at end and 1st quarter point; bracing at points 1 and 2; M_{max} at 2	$C_b = 1.52$
Case 4	Laterally braced at 1st and 2nd quarter points; bracing at points 2 and 3; M_{max} at 3	$C_b = 1.06$
Case 5	Laterally braced at 1st and 3rd quarter points; bracing at points 2 and 4; M_{max} at 3	$C_b = 1.03$

* Values from 1986 LRFD, Eq. 9.6.12 shown in parenthesis.



Nominal Moment Strength M_u as affected by C_b